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Manual of

PAVEMENT THICKNESS DESIGN

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PAVEMENT THICKNESS DESIGN MANUAL

by

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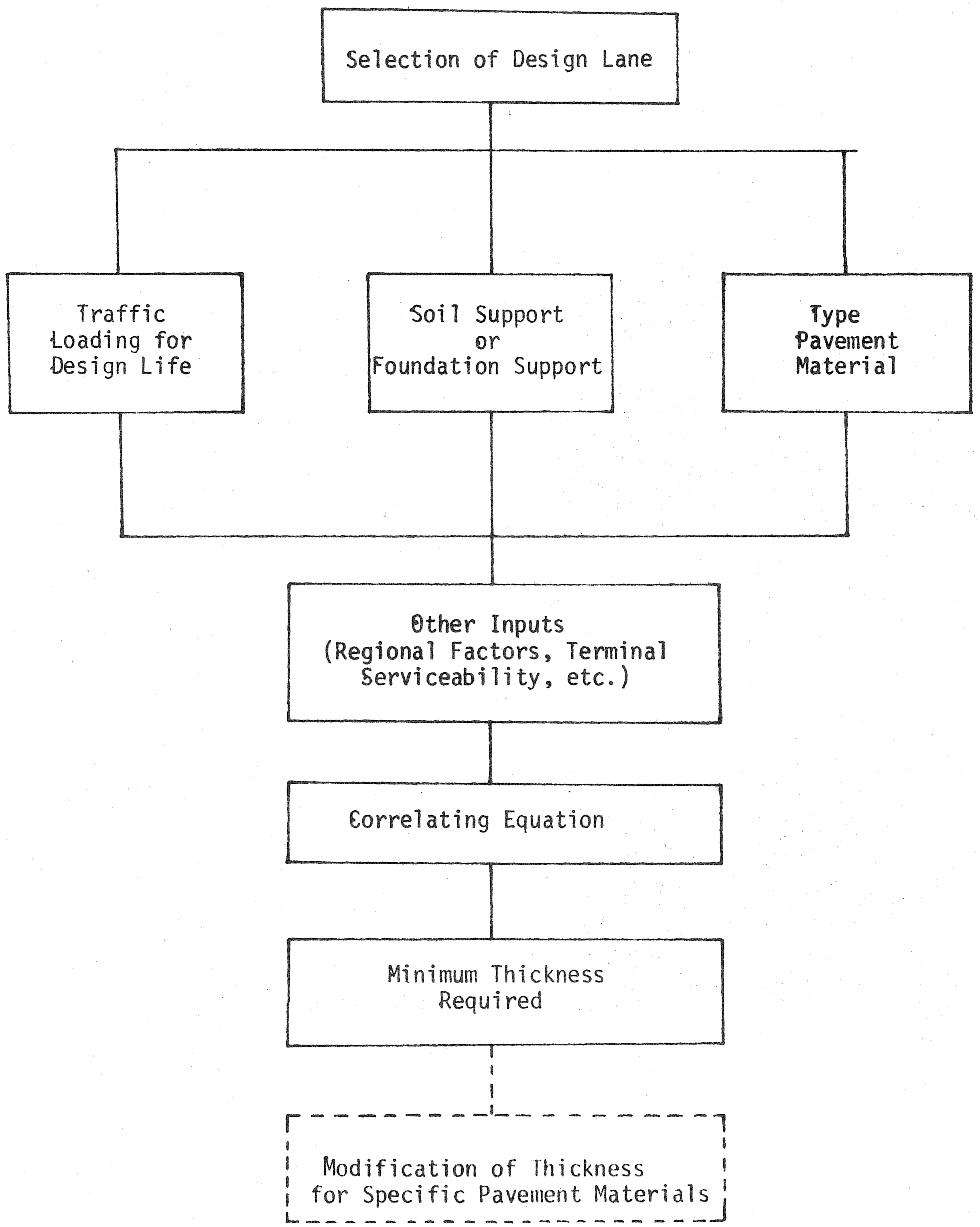
The intent of this manual is to provide a guide to pavement design. It is a supplement to procedures contained in accompanying materials and classroom presentations.

This manual should be used as a workbook and the class participants should add notes and comments as the course progresses.

Because the basis and practice of pavement design changes continuously, the contents in this manual are subject to change in the future. Like most things, it is important that all of us keep up with these new changes.

BASIS OF CONVENTIONAL PAVEMENT DESIGN METHODS



TYPES OF TRAFFIC LOADING INPUTS

Consider both Single and Tandem Axles

1. Equivalent 18-kip single axle loads

   Example: AASHTO Method, Basic Asphalt Institute Methods

2. Equivalent wheel loads

   Example: Basis of State of Idaho Method - 5,000 lb. wheel loads

3. Axle load groups w/o equivalencies

   Example: PCA Method for P.C. Concrete Pavements

4. Arrangement of Consistent Pattern of Equivalencies

   - when no large deviation from axle-load established pattern in region or state.

   Example: State of Idaho's Traffic Index

ADDITIONAL TRAFFIC LOADING INFORMATION AND CALCULATIONS

1. Current and Future Projection of:

   (a) Total traffic in design lane

   (b) Trucks in design lane

   (c) Truck axle or wheel weight distribution

   (d) Number of (c) - above expected for design life

2. Calculation of equivalencies if needed in thickness method used.

   Require Table of Equivalencies.

   Note: AASHTO 18-kip single axle equivalencies are dependent on:

   (a) Type of pavement: flexible or rigid,

   (b) Terminal serviceability index: 2.0 or 2.5,

   (c) Axle load type: single or tandem

   (d) Structural number (flexible) or concrete slab thickness (rigid).

TABLES OF EQUIVALENCIES

Example: AASHTO 18-kip single axles - from AASHTO Interim Guide

Flexible Pavement: Tables C.2-1 thru C.2-4

Rigid Pavement: Tables D.2-1 thru D.2-4

Assume flexible pavement, P_t = 2.0, SN = 3 (Table C.2-1)

Single axle load = 2.0 kips approximately equal to medium size passenger car. Equivalency per axle is .0002.

A "heavy" truck single axle load of 21-kips has an approximate equivalency per axle of 2.0.

Therefore, approximately 1 x 10^4 (ten thousand) single axle repetitions of a medium size passenger car are equivalent to one single repetition of a "heavy" truck 21-kip axle--in terms of pavement rate of damage (reduction of serviceablitity).

It is sensible to:

1. Build thickness of driveways to accommodate the loaded garbage truck,
2. Build thickness of residential streets to accommodate loaded garbage trucks and construction trucks,
3. Build thru highways to accommodate loaded garbage trucks, construction trucks, cargo-hauling trucks, etc.

When the highway becomes more arterial and intercity, more numbers of heavy trucks use it and therefore there are more equivalencies.

(Axle load x equivalency x number of axles)

Single axle load of 22-kips (Table C.2-1) has a 18-kip single axle load equivalency of 2.35. A tandem axle load of 41-kips

(Table C.2-2) has an approximate single axle load of 2.35. Or in another way, a 22-kip tandem axle load has a 18-kip single axle load equivalency of .18 or only: $\frac{.18}{2.35} \times 100 = 7.7\%$ of the 22-kip single axle load. (13% for rigid pavements).

If tandem-axle trucks cost the trucking industry more, the advantage is that they significantly reduce the rate of pavement damage for the same axle load.

CALCULATION OF SINGLE AXLE 18-KIP EQUIVALENCY

Use, as an example, the calculation procedure shown on pages 67-69, including traffic distribution Table C.2-6, of AASHTO Interim Guide. Calculation is for a 20-year design procedure. Road is to be two lanes, one lane each direction, and a flexible pavement.

The first step is to determine the current total vehicle count (both ways) that will most probably use the road, and then project this count for 20 years in the future. (This may not be extremely simple because a new, good road can attract more traffic than existing, poor roads in the origin-destination location).

Let us assume the initial ADT = 1500 vehicles per day (both directions) and 20-year projected ADT = 3000 vehicles per day (both directions). The percent of trucks = 22%, and $P_t = 2.5$, SN = 3.0.

1. Calculate the average ADT for the 20-yr. design period in both directions.

2. Calculate the 20-year ADT in one direction.

3. Calculate the average number of trucks per day in one direction for the 20-year design period.

4. Calculate the total number of trucks per day in one direction for the 20-year design period.

5. Using the loadometer station data in Table C.2-6, obtained on a nearby road, to represent the traffic distribution for the new pavement, calculate

   (a) The total 18-kip single axle load equivalents for the design lane for 20 years.

   (b) The average daily 18-kip single axle load equivalents for the design lane for 20 years.

The 18-kip single axle load equivalency, calculated, will be used later in the AASHTO procedure for the flexible pavement thickness design.

NOTE: The previous example was based on Table C.2-6 using the following distributions:

1. Ratio of numbers of single axles to tandem axles =

$$\frac{(512 + 536 + 239 + 1{,}453 + 279 + 106 + 43 + 4 + 3)}{(9 + 337 + 396 + 457 + 815 + 342 + 243 + 173 + 71 + 9 + 1)} = \frac{3175}{2853} = 1.11$$

2. Ratio of equivalent 18-kip single axles (single axles to tandem axles)

$$= \frac{459}{1368} = .336$$

3. The relative number of axles for each axle load in each distribution of single axles and tandem axles.

Roads serving different origin - destination characteristics and purposes may have axle-load distributions that are different than shown in Table C.2-6. It is therefore important to know the most probable axle-load distribution as possible, as well as total vehicle count. In many instances, state highway and local government agencies can furnish good estimates of axle loads and distributions for a particular locale.

The axle or wheel load equivalency, or the axle loads themselves, influences pavement thickness. Larger equivalencies mean thicker pavements. Another factor is the pavement foundation, in most cases, the natural soil support.

SOIL SUPPORT

The pavement thickness design procedure is usually dependent upon the type of soil support measurements.

Most of the time soil support is obtained by performing a mechanical - physical test on the soil in the laboratory. Sometimes this is done at the site location in the field. The objective of all soil support tests is to determine the internal resistance of the soil from application of external load. Each of the tests has its own peculiarities with respect to the type of internal resistance measured and with respect to the type of external load. A calculated number is obtained when the test results are applied to the particular devised calculation procedure. The following is an outline of some of the common soil support tests:

1. CBR. This test is a measure of shear resistance of a saturated, compacted soil (in a 6" diameter x 7" high mold) in the vicinity of a metal cylinder (3 in.^{2}) being pushed into the soil at a constant rate (.05 in/min) and up to a specified penetration of the cylinder (usually .1 inch). The stress (piston load divided by 3 in.^{2}) obtained is divided by a crushed gravel standard (1000 psi at .1 inch penetration); this ratio is obtained in percent.

Low % ratios (CBR's) of 2, 3, 4, 5% (or so) mean the soil is weak and a thicker pavement is needed as compared to % ratios of 15, 20 or 25. Silty clays and clayey silts are in the low % ratio ranges.

This test can also be performed on soil in the field (insitu).

CLASS NOTES:

2. R-Value. This is a test used in some of the Western States. It was originally developed with the Hveem measurement procedures in California.

The R-value test is indirectly a measure of the Poisson Effect of a soil--how much of a solid or a liquid-acting material it is. The prime parameters are P_v and P_h. P_v, a standard vertical load or pressure on a 4 inch diameter (2.5 inch thick) soil specimen, is equal to 2000 lbs. (approx. 160 psi). P_h, the dependent parameter, is the lateral pressure or load developed by the specimen at the 2000 lbs. (P_v) load. The load is placed on the specimen (4-inch diameter face) at a rate of .05 in. per min. The specimens usually have different water contents, compacted by constant compaction energy using a kneading compactor.

$$R = 100 - \frac{100}{\frac{2.5}{5}\left(\frac{P_v}{P_h} - 1\right) + 1}$$

D is equal to displacement turns, a correctional factor for specimen side unevenness. A Hveem Stabilometer is used for the testing.

$$R \text{ is proportional to } \frac{P_v}{P_h}.$$

Therefore, since P_v = standard constant, as P_h increses, R decreases (the soil behaves more fluid-like). A thicker pavement is therefore required. Low R-values of 10 - 20 reflect clay-like soils.

Since water content is varied, a plot of R vs. water content is made.

Each specimen is also tested under increasing load (stress) until enough water permeates from the specimen to cause electrical conductivity in an electrified chamber. The particular stress at which this occurs is called the exudation pressure. A plot of exudation pressure vs. water content is also made. The design water content is found from the plot at standard exudation pressure. Different agencies may require different compaction energies and different exudation pressures.

The design water content (exudation) is used in the R vs. water content plot to determine the design R-value.

Design R-values are used in pavement thickness design. For a given exudation pressure and kneading compaction energy, a given soil will have a given R-value.

The R-value test is a laboratory test.

CLASS NOTES:

3. Plate Bearing Test. This test is a field test on roadbed soil in-place. It is a measure of the stress, on a 30-inch diameter steel plate, required to penetrate the plate .05 inches into the soil.

The 30-inch plate is made more resistent to bending by stacking two smaller diameter plates above it. Load is applied by a jack reacting to a truck or trailer bumper-frame. Penetration is meausred by a dial gauge.

The stress for .05 inches of penetration is used to calculate

$$k = \frac{\text{stress for .05", psi}}{\text{.05 inches}}$$

$$= \text{subgrade "modulus"}$$

Low k, eg. 50 psi to 150 psi, indicates soft or weak subgrades.

When a treated or non-treated subbase is to be placed on the subgrade, the k on top of the subgrade is used for design. This k reflects both the subgrade and subbase resistance--a combined k. Combined k can be estimated knowing the subgrade k and the type and depth of the subbase.

Class handouts show how to calculate combined k.

k is mostly used in connection with portland cement concrete pavement thickness design.

4. Soil Classification. AASHTO soil classification (using Atterberg Limits and grain size) is used sometimes to evaluate a soil's support value. This is usually done regionally rather than universally, but still has limitations and is less desirable than mechanical tests.

A soil designated as A-6 provides, on the average, a weaker support than a soil designated as A-4, for example.

5. Resilient Modulus and Creep. These properties are developed from new types of repeated load tests on soil specimens in the laboratory. Usually a stress wave is produced on the soil in compression that reproduces field loading, eg. .2 sec. load duraton plus .8 sec. no-load rest, per cycle.

About 800 - 1000 cycles are applied to properly "exercise" the specimen before readings are taken. The deformation - strain response is measured as the dependent variable. Also, the base line change of deformation response is noted.

Sometimes a triaxial pressure cell is used with 1-3 psi confining pressure in order to simulate gravity confining stresses on the soil under the pavement or to hold up specimens that are non-cohesive. Repeated stress magnitudes (deviator stresses) range from 1 - 10 psi.

$$M_R = \frac{\text{magnitude of stress wave}}{\text{magnitude of recoverable strain wave}}$$

Creep = Base line non-recoverable strain per cycle.

M_R of 1,000 - 4,000 psi indicate weak soils or, more properly, soils of high resiliency which will cause high repeated pavement deflections unless the pavement section is adequately designed. High repeated pavement deflections cause early fatigue damage and cracking.

A high creep strain per cycle is indicative of pavement rutting. In some locations rutting is the primary cause of pavement failure; other locations show that high repeated deflections (early cracking) is the primary cause of pavement failure.

These values, which are in scientific units, are used directly in elastic or visco-elastic analysis procedures for calculation of critical stresse, strains and deflections.

6. <u>Correlations of Results between Soil Support Tests.</u>

Attempts have been made to correlate results from one test to those of another. The following are some correlations:

1. Soil Classification - CBR - R-value

   (Example: AASHTO Interim Guide Table C.3-1; Figure C.3-6)

2. R-value vs. k

   (Example: Idaho Div. of Hwys.)

3. R-value and M_R

   (Example: Chevron Research Corporation)

4. CBR and Elastic Modulus ($\approx M_R$)

   (Example: Asphalt Institute)

Note: Because each test type and results are different, it is difficult to obtain good correlations. Some correlations are subject to controversy.

7. Selection of Appropriate Numerical Value From Soil Support Test Results Obtained Within and Along Roadway.

AASHTO SOIL SUPPORT VALUE

The AASHTO soil support value is a quantitative number used to represent the "strength or load bearing" performance of subgrade soil.

There is no direct test procedure for this value. It is determined by correlation with the various conventional soil support test results.

Low values, eg. 1-4, indicate weak soils haveing CBR values in the range of 1-5, R-values in the range of 1-10, etc.

Examples of finding AASHTO soil support values are given in the AASHTO Interim Guide: Table c.3-1

Figures C.3-1, -2, -3, -4, -5, -6.

The AASHTO soil support values are used in the AASHTO flexible pavement thickness design procedure.

PAVEMENT MATERIALS

1. Rigidity or Stiffness

Portland Cement Concrete	[E ≈ 3,000,000 psi]
Dense-Graded Asphalt Concrete	[E ≈ 150,000 - 2,000,000 psi ~ f(Temp., water content, asphalt viscosity)]
Open-Graded Asphalt-Based Concrete	[E ≈ 15,000 - 150,000 psi ~ f(Temp., water content, asphalt viscosity, degree of openness and thickness of asph. film)]

Usually, the stiffer the pavement material the more sensitive the stress (and strain) distribution is with pavement thickness, within the pavement material and with its distribution to underlying materials.

A ¼ to ½-inch increase of thickness of portland cement concrete can result in satisfactory rather than unsatisfactory behavior. Approximately ½ to 1-inch increase of thickness of asphalt concrete may give equivalent results.

Stiffer pavement materials (eg. portland cement concrete) are not as sensitive to the soil support, ie. do not rely on as much soil support as flexible pavements. However, they require uniform soil support.

Stiffer pavement materials (eg. portland cement concrete, dense graded asphalt concrete) are usually more brittle and are subject to fatigue-cracking design considerations. Rational approaches frequently use critical stress analysis.

Low stiffness materials, conversely, are usually less brittle and are subject to plastic deformation design considerations. Rational approaches frequently use critical strain analysis.

There has been a trend over the past 20 years or so to make asphalt concrete stiffer. With the recent advance of asphalt emulsions and (also) open-graded asphalt concrete mixtures, these mixture types are found to be less stiff and more "flexible".

2. Durability

There are indications that less stiff, but sound, asphalt concrete mixtures are more "durable" but may produce wavy pavements; they require more support.

Stiff pavement materials (eg. dense-graded asphalt concrete and portland cement concrete) can crack. The propagation of cracks usually makes the pavement surface rough riding and unsightly.

Most cracks develop from: overstressing due to fatigue and from overstressing due to thermal contraction.

Fatigue is combated by designing adequate thicknesses to minimize stresses and strains at the bottom of the pavement layer. These stresses and strains have to be reduced if more traffic (trucks) are expected over the design life.

Thermal cracking is combated by making the pavement material less stiff, ie. use lower viscosity asphalt, greater film thicknesses, asphalts of low temperature susceptibility and of low hardening (oxidization) rate. This seems to allow the material to withstand the thermal contractive tensile stresses and strains more easily.

"Non-load" associated distress can occur, also.

Portland cement concrete is subjected to freeze-thaw damage, less so if air entrained, and to chemical reactions (eg. aggregate-alkali reactions and de-icing salt reactions). This is combated , in part, by careful selection of aggregates and cement, designing the concrete mixture as impermeable as possible, using good quality air entraining agents of correct amount. Slower hydration cements seem to produce less shrinkage cracks. Shrinkage cracking, mainly microcracks, are internal main crack initiators and therefore accelerate cracking rates on the pavement surface.

Asphalt concrete is subjected to hardening (and embrittlement) and to moisture damage. Hardening is combated by using low hardening rate asphalts and enough asphalt, with proper aggregate gradation, to reduce permeable voids down to the point where any further reduction will cause flushing or bleeding. Moisture damage can be a temporary ~ seasonal problem and/or a permanent problem. Moisture can enter from the pavement surface (downward) or from the underlying materials (upward). Moisture entry can reduce the stiffness of the asphalt concrete, changing the stress-strain distribution of the total pavement thickness and produce the effect of inadequate foundation support and rutting. When the asphalt concrete dries out, the original stiffness is regained. Permanent damage sometimes is known as stripping. Moisture displaces the asphalt from the aggregate. The displacement is permanent unless the mix is heated and reworked. Both stiffness and tensile strength are reduced, producing early cracking and raveling. This can be combated by using proper compatibility of aggregate and asphalt, lowest permeable voids as reasonable, and adding either asphalt anti-strip additives or treating the aggregate surfaces with lime or other chemical that retains asphalt.

Durability evaluations are best carred out in the laboratory under controlled conditions. Adequate personnel, equipment and funds should be utilized to make sure that material will be durable under realistic pavement conditions in the field. This is a good investment because the materials and construction costs may total hundreds of thousnads, even millions of dollars for large jobs.

3. Manufacture and Construction

Portland cement and asphalt concretes require aggregate and binder blending and control, hauling to construction site, a laydown process and, in the case of asphalt concrete, a compaction process. All processes require very careful attention in order to arrive at a uniform pavement material in-place having properties specifically desired at the correct thickness.

Moisture design in the laboratory and pavement thickness specified by a calculation procedure are not good enough by themselves. This has to be achieved physically in the field in order to complete the job. If what is desired might not or cannot be achieved in the field, then either more emphatic inspection and regulation is necessary or the specified requirements cannot be physically achieved by reasonable mortal men with typical construction equipment.

Can you give an example of each of the above situations?

4. Base and Subbase Materials

These materials are compacted and placed between the pavement material and subgrade when it is necessary to achieve a reduction of distributed stress from the bottom of the pavement layer to the soil subgrade. They are also used to provide a uniform bearing support layer for the pavement during and after construction. Sometimes economical reasons or certain distress sources lead one to minimize or eliminate the use of these materials; other times these criteria when applied, require one to use these materials.

They usually consist of layer granualar aggregates, treated or not treated with an economical binder.

CONVENTIONAL CORRELATING EQUATIONS FOR ASPHALT CONCRETE PAVEMENT

These correlating equations relate the inputs of:

Traffic Loading

Soil Support

and

Pavement Material Type

and produce, for example, a quantitative output of either:

1. Required structural member . . . to be satisfied by pavement layers (AASHTO procedure),
2. Equivalent gravel thickness . . . to be satisfied by pavement layers (Idaho, California, etc. procedures).

Other inputs may be climatic factors, terminal serviceablitiy, etc.

Correlating equations are developed from actual pavement performance (eg. surface condition) versus the pavement variables of materials, thicknesses, soil support, accumulated traffic loading. Many observations and combinations are needed. Equations are developed which divide good performance parameter combinations from the poor. Each input variable is evaluated.

WASHO and AASHO Test Roads are examples. The AASHTO correlation was determined from the AASHO Test Road results. The Idaho correlation was developed from the WASHO Test Road reuslts and with statewide and AASHO Test Road observations.

The AASHTO correlating equations and explanations (for flexible and rigid pavements) are listed in the appendices of the AASHTO Interim Guide.

The following sections explain pavement thickness design based on:

1. Correlating equations for asphalt concrete pavement (AASHTO and Idaho methods),

2. PCA method of stress-fatigue for portland cement concrete pavements, and

3. General outline of asphalt concrete pavement design based on resiliency (resilient modulus).

PREFACE TO PAVEMENT DESIGN METHODS: SERVICEABILITY

Serviceability is generally defined as the present abiltiy of a pavement to serve its traffic function.

It is a number when the traffic function is specified.

AASHTO defines serviceability:

- the ability at time of observation of a pavement to serve high-speed, high-volume automobile and truck traffic.

It is quantified in a range of 1-5, 5 being the highest or best, using a panel of highway specialists which rate the pavement's roughness, etc. -- or by using a formula based on physical measurements of the pavement's features. In the latter case, this is called "present serviceability index".

Terminal serviceability index is usually fixed at 2.0, or 2.5 in the case of high-type highways. The conditions of the road surface are then bad enought to overlay. Pavement designs are usually based on terminal serviceability index.

New constructed pavements should have a high present serviceability index, close to 5.0. Design of pavements is based on the gradual reduction of present serviceability index as traffic loading accumulates through the years. The pavement design life, therefore, is determined by the yearly rate of present serviceability index decrease, accumulated in total years, to the terminal serviceability index of 2.0 or 2.5.

New pavements are usually designed for a 20-year life. Sometimes portland cement concrete pavements are designed for a 40-year life.

Why do some pavements deteriorate sooner than expected, ie. 2.0 or 2.5 is reached too fast? Some reasons are:

1. Correlating equations used may not reflect the particular mode of

damage being developed in a particular locale with certain materials or traffic loading,

2. The traffic loading for 20 or 40 years is more than predicted,

3. The pavement materials are distinegrating or failing - eg. (unplanned): stripping, high amount of asphalt hardening, sudden subsidence of base or subgrade, unusually high rutting in wheel paths, surface raveling, freeze-thaw damage, chemical attack of portland cement past or aggregates, base aggregate degradation, insufficient removal of subsurface water -- plugging of drains, lack of uniformity -- quality control problems of pavement material, etc.,

4. The pavement layer thicknesses were constructed too low or there is too much ± variation of thickness, and

5. The soil support values used reflect only one section and a subtle but significant soil subgrade change really existed over the project length.

There is sometimes a need to design a pavement for lower serviceability -- based on intended use. Gravel roads and dirt roads are extreme examples. But sometimes they have to be upgraded as complaints and costs of transport rise. Here's an example:

> UNPAVED ROADS COST MONEY -- An Australian trucking company, operating a large fleet of long-haul vehicles in the Outback territory, reports alarmingly high cost differences between operating over unpaved and paved roads. Records maintained for 20 trucks traveling over more than 1,240,000 miles show that driving on unpaved (as opposed to paved) roads, increased tire costs by more than 500 percent, fuel operating costs by 20 percent, repairs and maintenance costs by 260 percent, and body overhaul expenditures by a massive 800 percent. What better argument could there be for paved roads?

"Stage construction" involves building a pavement that will deteriorate at a fast rate -- to a terminal serviceability index of 2.0 or 2.5 in, for example, 8 years, rather than 20 years. <u>But</u> within 8 years, more

pavement is placed, increasing the serviceability, so that the terminal serviceability in the future
is never below 2.0 or 2.5. (Mostly all pavements are "stage construction" if one looks at undetermined years in the future.) Ordinarily, stage construction is used when there is not enough capital to spend for a full 20-year design. Careful time-tables for overlay projects (adding more pavement) must be followed along with the planned capital on-hand for the future dates.

For most high-type roads, maintenance expenditures are so high when serviceability reaches 2.0 or 2.5, that it is more economical to add more pavement at that time under "rehabilitations or new construction" rather than letting the pavement deteriorate below serviceability of 2.0.

AASHTO METHOD OF ASPHALT CONCRETE PAVEMENT THICKNESS DESIGN

Refer to AASHTO Interim Guide for procedure. An explanation of the steps follows:

1. Obtain 18-kip single axle load equivalency based on daily repetitions or total repetitions in design lane.
2. Run conventional soil support test, obtain results, and convert to AASHTO soil support value. The soil support value is one of three basic inputs.
3. Determine terminal serviceability index: P_t = 2.0 or 2.5.
4. Determine regional factor: Section 2.4-2 AASHTO Interim Guide (probably 1.0 or greater; 5 is max. severe conditions.
5. Determine average 20-year equivalent 18-kip single axle load repetitions based on either: total 20-year average or daily average.
6. Calculate structural number (SN) by use of nomograph in Figures II-1 or II-2. Nomograph is solution to basic AASHTO correlating equation.

   SN is a quantitative number which represents the minimum required pavement stiffness made up of:

   (a) Thickness of layers, and

   (b) Stiffness of each individual layer's material selected.

   SN allows for one or more pavement layers, but the sum of (a) and (b) products above must be equal to or greater than SN.

   (a) , above, is in inches

   (b) , above, is a dimensionless number representing the cohesive stiffness of a layer material. Typical ranges are .1 to .5.
7. Select number of layers, pavement materials and trial thicknesses, (a) and (b) above for each layer, and calculate sum of (a) x (b) to

be equal to or greater than SN. Economic analysis of inplace materials and construction costs and/or specification requirements of materials and/or thicknesses will be needed to select most economical combination of (a) x (b).

Refer to Sections 2.4.3 and 2.4.4 (also Table II-3) for calculation of SN and materials' stiffnesses (layer coefficients).

$$SN = a_1 D_1 + a_2 D_2 + . . . + a_n D_n$$

Where : a = layer coefficients

D = layer trial thicknesses, inches.

(SN is basically a spring -linear series combination,)

CLASS NOTES:

AASHTO METHOD: ASPHALT CONCRETE
MINIMUM COST DESIGN

CLASS PROBLEM

The problem is to find the minimum thickness of pavement layers that would minimize the cost of a two-lane (12 ft. lane widths) flexible pavement 20 miles long. Design for terminal serviceability = P_t = 2.0.

Data: Repetition of 18-kip equivalents in design lane (average for 20-year design period) = 2,1117,000. = total.

Subgrade: R = 10.5, S = 4.9

Asphalt Concrete: Density as compacted = 142.0 pcf. Layer coefficient structural equivalency = .44. Depth equal or greater than 2 inches. Cost = $13.60/ton in place.

Granular Base: Density as compacted = 120.0 pcf. Layer coefficient structural equivalency = .14. Depth equal or greater than 4 inches if used. Cost = $3.85/ton in place.

Subbase: Density as compacted = 123.0 pcf. Coefficient structural equivalency = .11. Depth equal or greater than 4 inches if used. Cost = $2.95/ cu. yd.

Note: These costs may not be accurate from location to location and change with time. They are only examples for this problem.

The minimum cost thickness may include some or all of above material thicknesses on subgrade but must include at least asphalt concrete on subgrade. What is the design pavement thickness for minimum cost? What is the minimum cost?

PCA RATIONAL METHOD OF PORTLAND CEMENT CONCRETE THICKNESS DESIGN

Refer to procedure in PCA manual: Thickness Design for Concrete Pavements

1. Soil Support

2. Laboratory Modulus of Rupture Tests

3. Fatigue Property and Stress Ratio

4. Load Safety Factors

5. Load Stresses

6. Design Life

7. Calculation of Pavement Thickness

PCA METHOD: PORTLAND CEMENT CONCRETE THICKNESS DESIGN

CLASS PROBLEM

Problem: To calculate the minimum thicknesses (to nearest ½" above minimum) of p.c.c. pavement with and without a subbase using the following data:

1. Third-point M-R = 791 psi (high strength concrete).

2. Traffic count and axle distribution as given in example on pp. 65-66 of AASHTO Interim Guide for Design of Pavement Structures using Table C.2-6, 20-year design period with initial ACT = 4000 vpd (both directions) and projected 20-year ACT = 6000 vpd (both directions) 20% trucks, etc.

3. A maximum fatigue consumption of 100%

4. A subgrade k = 50 pci (average)

5. An optional untreated subbase of 4 inches thick.

6. High-type pavement with 1.2 LSF.

7. 100% trucks in design lane.

Traffic Calculation

M-R ____________

Subgrade k______________________ Subbase Thickness and Type____________________

Combined k______________________ Load Safety Factor____________________________

Trial Depth_____________________

Axle Loads kips	Axle Loads x L.S.F. kips	Stress psi	Stress Ratio	Allowable Repetitions	Expected Repetitions	Fatigue used, %
SINGLE AXLES						
TANDEM AXLES						

Total =

M-R ___________

Subgrade k____________________ Subbase Thickness and Type____________________

Combined k____________________ Load Safety Factor____________________

Trial Depth____________________

Axle Loads kips	Axle Loads x L.S.F. kips	Stress psi	Stress Ratio	Allowable Repetitions	Expected Repetitions	Fatigue used, %
SINGLE AXLES						
TANDEM AXLES						

Total =

M-R ____________

Subgrade k______________________ Subbase Thickness and Type______________________

Combined k______________________ Load Safety Factor____________________________

Trial Depth_____________________

Axle Loads kips	Axle Loads x L.S.F. kips	Stress psi	Stress Ratio	Allowable Repetitions	Expected Repetitions	Fatigue used, %
SINGLE AXLES						
TANDEM AXLES						

Total =

M-R ____________

Subgrade k____________________ Subbase Thickness and Type____________________

Combined k____________________ Load Safety Factor____________________

Trial Depth____________________

Axle Loads kips	Axle Loads x L.S.F. kips	Stress psi	Stress Ratio	Allowable Repetitions	Expected Repetitions	Fatigue used, %
SINGLE AXLES						
TANDEM AXLES						

Total =

M-R ____________

Subgrade k ____________ Subbase Thickness and Type ____________

Combined k ____________ Load Safety Factor ____________

Trial Depth ____________

Axle Loads kips	Axle Loads x L.S.F. kips	Stress psi	Stress Ratio	Allowable Repetitions	Expected Repetitions	Fatigue used, %
SINGLE AXLES						
TANDEM AXLES						

Total =

M-R ____________

Subgrade k____________________ Subbase Thickness and Type____________________

Combined k____________________ Load Safety Factor____________________

Trial Depth____________________

Axle Loads kips	Axle Loads x L.S.F. kips	Stress psi	Stress Ratio	Allowable Repetitions	Expected Repetitions	Fatigue used, %
SINGLE AXLES						
TANDEM AXLES						

Total =

M-R ____________

Subgrade k______________________ Subbase Thickness and Type____________________

Combined k______________________ Load Safety Factor____________________________

Trial Depth_____________________

Axle Loads kips	Axle Loads x L.S.F. kips	Stress psi	Stress Ratio	Allowable Repetitions	Expected Repetitions	Fatigue used, %
SINGLE AXLES						
TANDEM AXLES						

Total =

PORTLAND CEMENT CONCRETE PAVEMENT: SLAB AND JOINT DESIGN

A continuously, unreinforced, poured concrete pavement will crack in a regular pattern relatively fast. The cracking will be irregular and unsightly.

This cracking is due to the shrinkage of concrete as it hydrates and as it cools. The tendency to shrink is resisted by friction between the concrete and the subgrade soil (or subbase material). This resistance produces internal tensile stresses in the concrete. These stresses, in magnitude,are proportional to the tensile strain build-up. The tensile strain builds up with increasing length of the concrete. At some length point, the stresses exceed the tensile strength of the concrete and the concrete cracks.

Since the concrete will crack, it is desirable to force these contraction cracks to be either:

1. Regular and uniform, filled with good quality joint sealer, or
2. Tiny, held together by reinforcement.

Regular and uniform cracks are produced by sawing transversely the <u>fresh</u> concrete at slab lengths of random lengths of 10, 12, 11, 13 foot, etc. for example. The transverse joints are also sawed on the skew to minimize tire-vehicle vibrations. The longitudinal joint (between lanes) is also sawed. Saw depths are about ¼ of the concrete slab depth.

Polymer or bituminous sealer is placed in the sawed joints. Sometimes plastic strip sealer placed by machine is also used.

This type of short-slab concrete pavement needs no internal steel reinforcement because additional cracks between the joints should not form under properly designed pavement. These pavements are popular.

Slabs with joints spaced 30-60 feet, for example, are sometimes used. They require steel reinforcement because cracks form between the joints. These types of intermediate - large slab concrete pavements are losing popularity.

A second popular concrete pavement is the continuously reinforced concrete pavement. Reinforcement steel is used to hold the cracks tightly together so that foreign matter will not enter, and keeps the pavement smooth. These pavements are popular because there is an opinion that these very long "slab" lengths give better life and they are smoother riding because of no short-distance joints.

All concrete pavements require construction joints when the job shuts down for the day. These joints can be designed as contraction joints or as closed load-transfer joints.

The construction joint as a contraction joint will require free moving dowels to aid in load transfer.

The construction joint as a closed load-transfer joint will require deformed tie bars. A contraction joint will need to be placed on either side of the construction joint for short-slab concrete pavements. A contraction joint is not needed for continuously reinforced pavement.

PORTLAND CEMENT CONCRETE PAVEMENT: DOWELS, TIE BARS AND REINFORCEMENT

The price of steel and labor of placement requires careful consideration of when and where to place steel.

1. Dowels

Dowels are smooth bars which are used to transfer transverse pavement edge wheel loads from one slab to the next. They are not bonded to the concrete. They must be free to move during contraction - expansion of the contraction joint. Therefore they must also be placed parallel and horizontal to the slab longitudinal axis.

Dowels can also be placed in a construction joint if the construction joint is intended to be a contraction joint.

The benefit of dowels is currently open to debate. In contraction joints, the upper ¼ slab depth sawed, the lower 3/4 of the slab depth the cracks in a random pattern around large concrete aggregates. This produces a vertical irregular joint in the lower 3/4'ths which appears to have enough friction interlock to produce the required load transfer.

2. Tie Bars

Tie bars are deformed reinforcing bars which are required to bond with the concrete. They are placed transversely at the longitudinal joints between lane slabs and between the outside lane and concrete shoulder slabs. Their major function is to keep the slabs from pulling away from each other transversely across the pavement cross-section.

Another reason for the bars is their helpful stress transfer benefits between the outside of the outer lane slab and the concrete slab shoulder, when existing. This appears to increase pavement life.

Tie bars are also used in construction joints, to tie and hold them together, when the construction joint is not intended to be a contraction joint.

The longitudinal joint slab separation resistance benefit of tie bars is also open to debate. There is now a growing opinion that the modern constant slope cross-sections of concrete pavements and subgrade (or subbase) friction produce an adequate situation, by themselves, for resisting slab separation.

3. Internal Reinforcement

Internal reinforcement is used in long slabs and continuously reinforced concrete pavement to hold contraction cracks together in order to maintain a smooth riding surface.

It is not intended to take bending tensile stresses, such as in ordinary reinforced concrete, because:

(a) The concrete slab is designed so that its flexure stresses are below its flexure strength and the slab is in contact with the subgrade soil (or subbase), and

(b) Reinforcement steel is usually placed near the neutral axis of slab bending for economical and practical reasons of: aggregate interference in lower sections of the slab and variations in height control on the job.

Selection of the type of reinforcement steel is a supply-economy problem. These considerations may lead one to choose the short slab pavement. However, the seemingly longer serviceability benefit of continuously reinforced pavement may outweigh the additional cost when steel reinforcement is being considered.

PORTLAND CEMENT CONCRETE PAVEMENT: CALCUALTION AND PLACEMENT OF DOWELS, TIE BARS AND INTERNAL REINFORCEMENT

CLASS PROBLEM

Refer to PCA handouts on: Dowels, Tie Bars, and Reinforcements.

Using the slab thicknesses calculated previously in the PCA method of pavement design,

(a) Sketch the slab lengths, and calculate and show the steel requirements, if any, (include a construction joint and concrete shoulder) for a short-slab pavement.

(b) Calculate and show the steel requirements, if any, (include a constructions joint and concrete shoulder) for a continuously reinforced pavement.

RATIONAL ASPHALT CONCRETE PAVEMENT DESIGN PROCEDURES

Current developing methods use:

(a) elastic stress-strain computational procedures which calculate stresses-strain-displacements due to the cyclic wheel load rebound response. Resilient modulus, M_R, and, sometimes, Poisson's Ratio, is used for each material layer. Thicknesses of layers are assumed for a particular solution. Trial thicknesses eventually iterate to a solution with stresses, strains and displacements that are allowable -- within specifications for the design traffic period.

A design wheel load and tire pressure is used. The area of tire-pavement contact is determined by dividing the wheel load by the tire pressure -- an approximate method.

Examples of currently used procedures are: Chevron 5-L, Chevron 15-L and Odemark programs. There are some programs based on finite element theory.

A typical Chevron 5-L program output is shown on the following three pages for high wheel load. The computer surface deflection, tensile strain at bottom of asphalt concrete and vertical strain on tope of subgrade tend to be critical. These computed values are compared to maximum allowables (specifications) for the traffic design period.

(b) Visco-elastic stress-strain computational procedures utilize the creep (non-recoverable) as well as the recoverable quasi-elastic response of the pavemen materials. FHWA has developed the VESYS II M program and is being evaluated. The program is also supposed to predict the amount of cracking with time as well as the rutting. Utah is presently one state that is evaluating the FHWA program. Materials Research and Development in Oakland, CA. is developing the NCHRP structural sybsystem program which also includes low temperature thermal cracking predictions.

CHEV 5-L

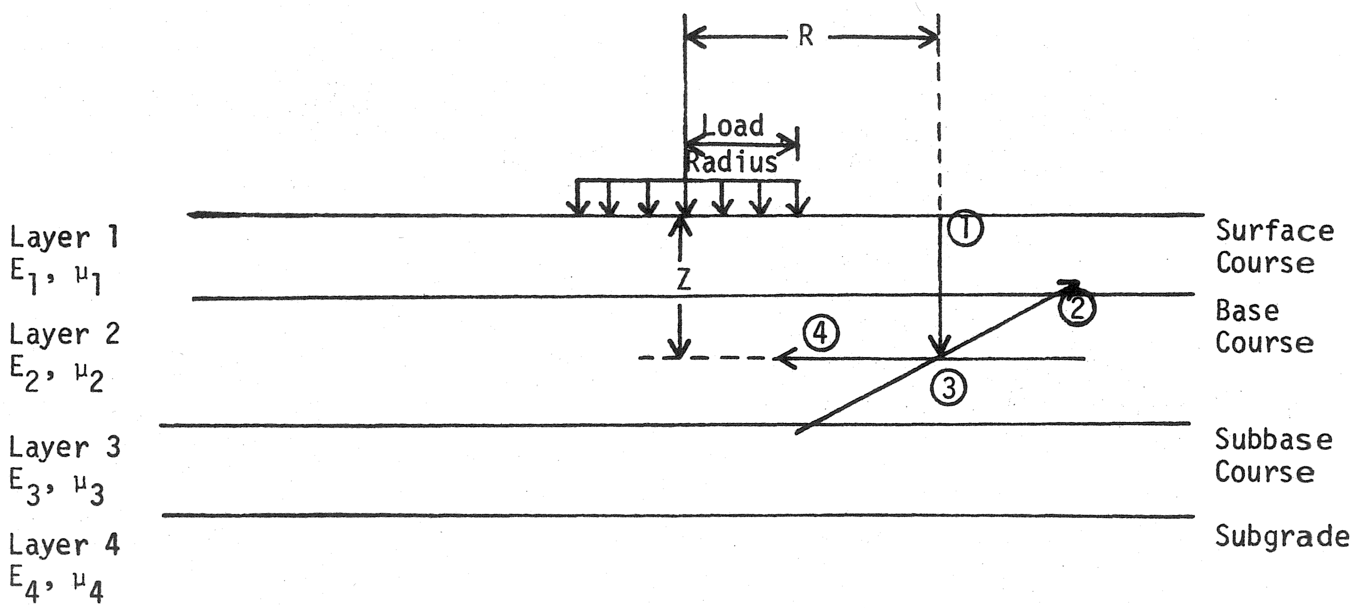


FIGURE 1

PROFILE SHOWING STRESSES AT A POINT IN A LAYERED SYSTEM

At any radius, R, and depth, Z, on the system:

1 is the vertical stress at the point

2 is the tangential stress at the point

3 is the radical stress at the point

4 is the shear stress at the point.

The sum of 1, 2, 3, and 4 is the bulk stress at the point.

******************** ANALYSIS OF COLORADO A.C. PAVEMENT DESIGN DRY CONDITION ******************** PAGE 1

THE PROBLEM PARAMETERS ARE

TOTAL LOAD.. 11000.00 LBS

TIRE PRESSURE.. 70.00 PSI

LOAD RADIUS.. 7.07 IN.

LAYER 1 HAS MODULUS 352000. POISSONS RATIO 0.350 AND THICKNESS 8.00 IN.
LAYER 2 HAS MODULUS 30000. POISSONS RATIO 0.400 AND THICKNESS 4.00 IN.
LAYER 3 HAS MODULUS 15000. POISSONS RATIO 0.400 AND THICKNESS 10.00 IN.
LAYER 4 HAS MODULUS 3000. POISSONS RATIO 0.450 AND IS SEMI-INFINITE.

		STRESSES						DISPLACEMENT		STRAINS			
R	Z	VERTICAL	TANGENTIAL	RADIAL	SHEAR	BULK		VERTICAL		RADIAL	TANGENTIAL	BULK	
0.0	0.0	-7.000E+01	-1.676E+02	-1.676E+02	0.0	-4.052E+02	*	4.142E-02	*	-2.399E-04	-2.399E-04	-3.453E-04	SLOW
0.0	0.5	-6.955E+01	-1.455E+02	-1.455E+02	0.0	-3.605E+02	*	4.147E-02	*	-1.995E-04	-1.995E-04	-3.072E-04	SLOW
0.0	1.0	-6.828E+01	-1.244E+02	-1.244E+02	0.0	-3.170E+02	*	4.151E-02	*	-1.617E-04	-1.617E-04	-2.702E-04	SLOW
0.0	1.5	-6.615E+01	-1.042E+02	-1.042E+02	0.0	-2.746E+02	*	4.153E-02	*	-1.267E-04	-1.267E-04	-2.340E-04	
0.0	2.0	-6.320E+01	-8.505E+01	-8.505E+01	0.0	-2.333E+02	*	4.153E-02	*	-9.421E-05	-9.421E-05	-1.988E-04	
0.0	2.5	-5.951E+01	-6.677E+01	-6.677E+01	0.0	-1.930E+02	*	4.152E-02	*	-6.413E-05	-6.413E-05	-1.645E-04	
0.0	3.0	-5.516E+01	-4.928E+01	-4.928E+01	0.0	-1.537E+02	*	4.149E-02	*	-3.615E-05	-3.615E-05	-1.310E-04	
0.0	3.5	-5.030E+01	-3.245E+01	-3.245E+01	0.0	-1.152E+02	*	4.146E-02	*	-9.912E-06	-9.912E-06	-9.819E-05	
0.0	4.0	-4.508E+01	-1.613E+01	-1.613E+01	0.0	-7.734E+01	*	4.141E-02	*	1.503E-05	1.503E-05	-6.592E-05	
0.0	4.1	-4.400E+01	-1.291E+01	-1.291E+01	0.0	-6.983E+01	*	4.140E-02	*	1.990E-05	1.990E-05	-5.951E-05	
0.0	4.2	-4.292E+01	-9.707E+00	-9.707E+00	0.0	-6.233E+01	*	4.139E-02	*	2.475E-05	2.475E-05	-5.313E-05	
0.0	4.3	-4.183E+01	-6.510E+00	-6.510E+00	0.0	-5.485E+01	*	4.138E-02	*	2.957E-05	2.957E-05	-4.675E-05	
0.0	4.4	-4.074E+01	-3.321E+00	-3.321E+00	0.0	-4.738E+01	*	4.137E-02	*	3.438E-05	3.438E-05	-4.038E-05	
0.0	4.5	-3.965E+01	-1.387E-01	-1.387E-01	0.0	-3.993E+01	*	4.136E-02	*	3.917E-05	3.917E-05	-3.403E-05	
0.0	4.6	-3.856E+01	3.038E+00	3.038E+00	0.0	-3.248E+01	*	4.135E-02	*	4.395E-05	4.395E-05	-2.768E-05	
0.0	4.7	-3.747E+01	6.212E+00	6.212E+00	0.0	-2.504E+01	*	4.134E-02	*	4.872E-05	4.872E-05	-2.134E-05	
0.0	4.8	-3.637E+01	9.383E+00	9.383E+00	0.0	-1.761E+01	*	4.133E-02	*	5.350E-05	5.350E-05	-1.501E-05	
0.0	4.9	-3.529E+01	1.255E+01	1.255E+01	0.0	-1.018E+01	*	4.131E-02	*	5.827E-05	5.827E-05	-8.674E-06	
0.0	5.0	-3.421E+01	1.573E+01	1.573E+01	0.0	-2.750E+00	*	4.130E-02	*	6.305E-05	6.305E-05	-2.344E-06	
0.0	5.1	-3.313E+01	1.890E+01	1.890E+01	0.0	4.678E+00	*	4.129E-02	*	6.785E-05	6.785E-05	3.987E-06	
0.0	5.2	-3.206E+01	2.208E+01	2.208E+01	0.0	1.211E+01	*	4.128E-02	*	7.266E-05	7.266E-05	1.032E-05	
0.0	5.3	-3.100E+01	2.527E+01	2.527E+01	0.0	1.954E+01	*	4.126E-02	*	7.750E-05	7.750E-05	1.666E-05	
0.0	5.4	-2.996E+01	2.847E+01	2.847E+01	0.0	2.698E+01	*	4.125E-02	*	8.236E-05	8.236E-05	2.300E-05	
0.0	5.5	-2.892E+01	3.168E+01	3.168E+01	0.0	3.443E+01	*	4.123E-02	*	8.725E-05	8.725E-05	2.934E-05	
0.0	5.6	-2.790E+01	3.489E+01	3.489E+01	0.0	4.189E+01	*	4.122E-02	*	9.218E-05	9.218E-05	3.570E-05	
0.0	5.7	-2.689E+01	3.813E+01	3.813E+01	0.0	4.936E+01	*	4.120E-02	*	9.714E-05	9.714E-05	4.207E-05	
0.0	5.8	-2.590E+01	4.138E+01	4.138E+01	0.0	5.685E+01	*	4.119E-02	*	1.022E-04	1.022E-04	4.845E-05	
0.0	5.9	-2.493E+01	4.464E+01	4.464E+01	0.0	6.435E+01	*	4.117E-02	*	1.072E-04	1.072E-04	5.484E-05	
0.0	6.0	-2.398E+01	4.793E+01	4.793E+01	0.0	7.187E+01	*	4.116E-02	*	1.123E-04	1.123E-04	6.125E-05	
0.0	6.1	-2.305E+01	5.123E+01	5.123E+01	0.0	7.941E+01	*	4.114E-02	*	1.175E-04	1.175E-04	6.768E-05	
0.0	6.2	-2.214E+01	5.456E+01	5.456E+01	0.0	8.698E+01	*	4.112E-02	*	1.228E-04	1.228E-04	7.413E-05	
0.0	6.3	-2.126E+01	5.792E+01	5.792E+01	0.0	9.457E+01	*	4.111E-02	*	1.281E-04	1.281E-04	8.060E-05	

***************** ANALYSIS OF COLORADO A.C. PAVEMENT DESIGN DRY CONDITION ***************** PAGE 2

P	Z	STRESSES VERTICAL	STRESSES TANGENTIAL	STRESSES RADIAL	STRESSES SHEAR	STRESSES BULK		DISPLACEMENT VERTICAL		STRAINS RADIAL	STRAINS TANGENTIAL	STRAINS BULK
0.0	6.4	-2.040E+01	6.130E+01	6.130E+01	0.0	1.022E+02	*	4.109E-02	*	1.335E-04	1.335E-04	8.710E-05
0.0	6.5	-1.958E+01	6.471E+01	6.471E+01	0.0	1.098E+02	*	4.107E-02	*	1.390E-04	1.390E-04	9.362E-05
0.0	6.6	-1.878E+01	6.815E+01	6.815E+01	0.0	1.175E+02	*	4.105E-02	*	1.445E-04	1.445E-04	1.002E-04
0.0	6.7	-1.801E+01	7.163E+01	7.163E+01	0.0	1.252E+02	*	4.103E-02	*	1.502E-04	1.502E-04	1.067E-04
0.0	6.8	-1.727E+01	7.514E+01	7.514E+01	0.0	1.330E+02	*	4.101E-02	*	1.559E-04	1.559E-04	1.134E-04
0.0	6.9	-1.657E+01	7.868E+01	7.868E+01	0.0	1.408E+02	*	4.099E-02	*	1.618E-04	1.618E-04	1.200E-04
0.0	7.0	-1.590E+01	8.227E+01	8.227E+01	0.0	1.486E+02	*	4.097E-02	*	1.677E-04	1.677E-04	1.267E-04
0.0	7.1	-1.527E+01	8.589E+01	8.589E+01	0.0	1.565E+02	*	4.095E-02	*	1.738E-04	1.738E-04	1.334E-04
0.0	7.2	-1.468E+01	8.956E+01	8.956E+01	0.0	1.644E+02	*	4.093E-02	*	1.800E-04	1.800E-04	1.402E-04
0.0	7.3	-1.413E+01	9.328E+01	9.328E+01	0.0	1.724E+02	*	4.091E-02	*	1.863E-04	1.863E-04	1.470E-04
0.0	7.4	-1.363E+01	9.704E+01	9.704E+01	0.0	1.805E+02	*	4.088E-02	*	1.927E-04	1.927E-04	1.539E-04
0.0	7.5	-1.316E+01	1.009E+02	1.009E+02	0.0	1.885E+02	*	4.086E-02	*	1.993E-04	1.993E-04	1.607E-04
0.0	7.6	-1.275E+01	1.047E+02	1.047E+02	0.0	1.967E+02	*	4.084E-02	*	2.060E-04	2.060E-04	1.676E-04
0.0	7.7	-1.238E+01	1.086E+02	1.086E+02	0.0	2.049E+02	*	4.081E-02	*	2.129E-04	2.129E-04	1.746E-04
0.0	7.8	-1.206E+01	1.126E+02	1.126E+02	0.0	2.131E+02	*	4.079E-02	*	2.199E-04	2.199E-04	1.817E-04
0.0	7.9	-1.179E+01	1 1.166E+02	1.166E+02	0.0	2.215E+02	*	4.076E-02	*	2.271E-04	2.271E-04	1.888E-04
0.0	-8.0	-1.158E+01	$\boxed{1.207E+02}$ CRIT.	1.207E+02	0.0	2.299E+02	*	4.073E-02	*	2.344E-04	2.344E-04	1.959E-04
0.0	8.0	-1.158E+01	4.004E+00	4.004E+00	0.0	-3.570E+00	*	4.073E-02	*	2.344E-04	2.344E-04	-2.380E-05
0.0	8.1	-1.139E+01	4.098E+00	4.098E+00	0.0	-3.195E+00	*	4.068E-02	*	2.338E-04	2.338E-04	-2.130E-05
0.0	8.2	-1.121E+01	4.193E+00	4.193E+00	0.0	-2.820E+00	*	4.063E-02	*	2.333E-04	2.333E-04	-1.880E-05
0.0	8.3	-1.103E+01	4.289E+00	4.289E+00	0.0	-2.447E+00	*	4.059E-02	*	2.328E-04	2.328E-04	-1.631E-05
0.0	8.4	-1.085E+01	4.386E+00	4.386E+00	0.0	-2.075E+00	*	4.054E-02	*	2.324E-04	2.324E-04	-1.383E-05
0.0	8.5	-1.067E+01	4.484E+00	4.484E+00	0.0	-1.703E+00	*	4.049E-02	*	2.320E-04	2.320E-04	-1.135E-05
0.0	-22.0	-1.997E+00	5.671E+00	5.671E+00	0.0	9.345E+00	*	3.463E-02	*	2.801E-04	2.801E-04	~~1.246E-04~~
0.0	22.0	-1.997E+00	-1.062E-01	-1.062E-01	0.0	-2.210E+00	*	3.463E-02	*	2.801E-04	2.801E-04	-7.365E-05 3 CRIT.

1 Maximum Tensile Stress at the Bottom of Asphalt Concrete Layer

2 PAVement Deflection (inches)

3 Bulk Strain at the top of Subgrade

(Vertical Compressive strain = Bulk Strain — (Tangential Strain + Radial Strain)

(Vertical Compressive Strain will be a negative number)

$$\mathcal{E}_v = -7.365\times10^{-5} - (2\times2.801\times10^{-4}) = -6.339\times10^{-4}$$

54

ELASTIC LAYER SURFACE DISPLACEMENT BY ODEMARK

A simplified elastic approach for calculation of elastic response pavement surface displacement was developed by Odemark. This calculation can be done by the conventional calculator.

An example calculation is given on the following pages.

The maximum allowable amount of surface displacement is based on:

(a) Traffic loading (repetitions, etc.)

(b) Type of surface deflection correlation to good pavement performance: Benkelman Beams (more than one type), Dynafleet, etc.

(c) Pavement material or mix type (dense and stiff vs. less stiff).

For higher traffic loading, there is agreement that surface deflection should be between .015 inches to .040 inches, depending on (a), (b), and (c) above.

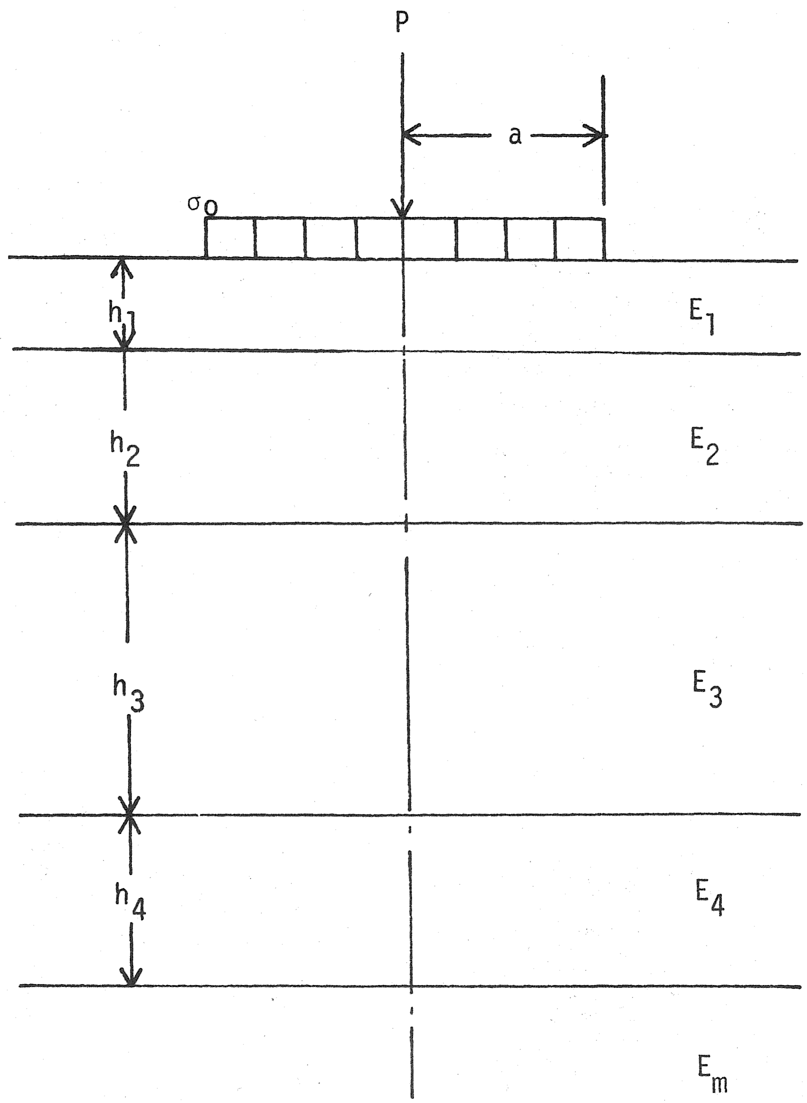
Figure IV-8 in AASHTO Interim Guide, for example, gives the Asphalt Institute's version of max. allowable deflections (refer to zero overlay thickness deflections).

ODEMARK I

gives displacements ≈ displ. when Chev 5L vert. ε on sg. is satisfied.

An alternate approach for the calculation of surface displacements for a three or more layered elastic system is by equations developed by Odemark.

For a system that can be represented as shown:



Where P = Wheel load = $\pi a^2 \sigma_o$

a = radius of uniformly loaded area

σ_0 = tire pressure, uniformly distributed over a circular area

E_1 = Young's Modulus of layer 1

E_2 = Young's Modulus of layer 2

E_3 = Young's Modulus of layer 3

E_4 = Young's modulus of layer 4

E_m = Young's modulus of the subgrade

h_1 = thickness of layer 1

h_2 = thichness of layer 2

h_3 = thickness of layer 3

h_4 = thickness of layer 4

layer 5 is assumed to semi-infinite

The vertical surface displacement can be obtained by the following equation:

eq.1 $$w \pm \frac{3\sigma_o a}{2}\left(\frac{1-\cos\alpha_1}{E_1} + \frac{\cos\beta_1 - \cos\alpha_2}{E_2} + \frac{\cos\beta_2 - \cos\alpha_3}{E_3} + \frac{\cos\beta_3}{E_m}\right)$$

where w is the vertical surface displacement, and

eq.2 $$\cos\alpha_1 = \frac{a}{\sqrt{a^2 + q_{e1}{}^2}}$$

eq.3 $$\cos\alpha_2 = \frac{a}{\sqrt{a^2 + (h_{e1} + q_{e2})^2}}$$

eq.4 $$\cos\alpha_3 = \frac{a}{\sqrt{a^2 + (h_{e1} + h_{e2} + q_{e2})^2}}$$

eq.5 $$\cos\beta_1 = \frac{a}{\sqrt{a^2 + h_{e1}{}^2}}$$

eq.6 $$\cos\beta_2 = \frac{a}{\sqrt{a^2 + (h_{e1} + h_{e2})^2}}$$

eq.7 $$\cos\beta_3 = \frac{a}{\sqrt{a^2 + (H_{e1} + h_{e2} + h_{e3})^2}}$$

$$q_{e1} = 0.9 \times h_1$$

$$q_{e2} = 0.9 \times h$$

eq. 8 $$h_{e1} = 0.9 \times h_1 \times \sqrt[3]{\frac{E_1}{E_m}}$$

eq. 9 $$h_{e2} = 0.9 \times h_2 \times \sqrt[3]{\frac{E_2}{E_m}}$$

eq. 10 $$h_{e3} = 0.9 \times h_3 \times \sqrt[3]{\frac{E_3}{E_m}}$$

0.9 is a reduction factor for flexible pavements determined by Odemark. Poisson's ratios for all layers is assumed to be 0.5.

Example: Given: P = 7,500 lb.

σ_o = 80 psi

E_1 = 900,000 psi

E_2 = 15,000 psi

E_m = 3,000 psi

h_1 = 4.0

h_2 = 18.0 in.

Find: The vertical surface displacement

E_{ac} Idaho ≈ 400,000 psi

Good crushed stone, $E \simeq$ 25,000 psi

Solution: Know that $P = \pi a^2 \sigma_o$, therefore

$$a = \sqrt{\frac{P}{\pi \sigma_o}}$$

$$= 5.46 \text{ in.}$$

$$q_{e1} = 0.9 \times 4.0 \text{ in.} = 3.6 \text{ in.}$$

$$q_{e2} = 0.9 \times 18.0 \text{ in.} = 16.2 \text{ in.}$$

from egs. 8 and 9, $h_{e1} = 0.9 \times 4 \times \sqrt[3]{\frac{900,000}{3,000}}$

$= 24.099$ in.

$h_{e2} = 0.9 \times 18.0 \times \sqrt[3]{\frac{15,000}{3,000}}$

$= 27.70$ in.

substitution into eqs. 2 - 7 yields:

$$\cos_{1} = 0.83486$$

$$\cos_{2} = 0.13426$$

$$\cos_{1} = 0.22096$$

$$\cos_{2} = 0.104825$$

substitution into eq. 1 yields the desired vertical displacement:

$$w = \frac{3 \times 80 \times 5.46}{2} \left(\frac{1-0.83486}{900,000} + \frac{0.22096 - 0.13426}{15,000} + \frac{0.104825}{3,000} \right) = \underline{\underline{0.0273 \text{ in.}}}$$

For ease in calculations the following computer program can be used for a three layer system with $\mu = 0.5$.

SOURCE LISTING OF COMPUTER PROGRAM TO OBTAIN VERTICAL SURFACE DISPLACEMENTS BY ODEMARK'S METHOD FOR A THREE-LAYER SYSTEM.*

```
STMT  LEV  NT

  1         0    PROGNAME: PROC OPTIONS (MAIN);
  2    1    0    DEFAULT RANGE (*) VALUE (FLOAT DECIMAL (11));
  3    1    0    DCL PI INIT(3.14.59);
  4    1    0    GET DATA (N);
  5    1    0    DD: DO I-1 TO N;
  6    1    1    GET DATA (L, SIGMA, E1, E2, E3, H1, H2);
  7    1    1    CONSTANT 1=0.9;
  8    1    1    GE1=CONSTANT_1 * H1;
  9    1    1    GE2=CONSTANT_1 * H2;
 10    1    1    HE1=GE1 8 (E1/E3) ** 0.333333;
 11    1    1    HE2=GE2 * (E2/E3) ** 0.333333;
 12    1    1    A=SQRT(L/(SIGMA * PI));
 13    1    1    COSA1=A/SQRT (A * A + GE1 * GE1);
 14    1    1    COSA2=A/SQRT (A * A + ((HE1 + GE2) ** 2));
 15    1    1    COSB1=A/SQRT (A * A + HE1 + HE1);
 16    1    1    COSB2=A/SQRT (A * A + ((HE1 + HE2) ** 2));
 17    1    1    CONSTANT 2=(3 * SIGMA * A)/2.0;
 18    1    1    DELTA=CONSTANT_2 * (((1-COSA1)/E1) + (( COSB1-
                 COSA2)/E2) + (COSB2/E3));
 19    1    1    PUT SKIP DATA (L);
 20    1    1    PUT SKIP DATA (SIGMA);
 21    1    1    PUT SKIP DATA (E1);
 22    1    1    PUT SKIP DATA (E2);
 23    1    1    PUT SKIP DATA (E3);
 24    1    1    PUT SKIP DATA (H1);
 25    1    1    PUT SKIP DATA (H2);
 26    1    1    PUT SKIP (2) DATA (DELTA);
 27    1    1    END DD;
 28    1    1    END PROGNAME;
```

* Written in PL/I language

ELASTIC LAYER SURFACE DISPLACEMENT ERRORS

Existing asphalt concrete pavements need periodic evaluation of their serviceability. If deflection begins to increase rapidly, rate of decrease of serviceability increases and the pavement must be programmed for an overlay sooner than expected.

When predicting deflections by computational methods, one must keep in mind the following:

(a) Computational procedures assumptions,

(b) Test equipment accuracy for materials testing to determine module, especially subgrade modulus,

(c) Layer thickness variations due to construction practice,

(d) Layer modulus variations along roadway due to moisture intrusion, variations, density variations, etc.

The following is a brief discussion of variations:

The accuracy with which the Benkelman Beam can measure vertical displacement of asphalt pavement surfaces is determined to be about ± 10%. This is termed the "expected measurability error" of the Benkelman Beam.

The accuracy to which subgrade modulus should be determined, to limit variations in vertical displacements to the expected measurability error, is a function of variations in the other input variables of the system. The greater the number of variables due to construction technique, for example, the more precisely the subgrade modulus should be determined. This is shown to be particularly true for the full-depth asphalt concrete system when the asphalt concrete moduli and thicknesses are assumed to vary ± 10% each.

Even small variations in subgrade modulus, in conjunction with a ± 10% variation in some other variable(s), can result in significant variations in vertical surface displacements, as shown in the following table. In general, a measurement of subgrade modulus may be in 10% error. This can lead to a 14-20% variation of pavement deflection.

Variations in "as-placed" properties of asphalt pavement systems do exist. In order to design for more realistic situations these variations should be considered.

Variations in Input Variables = ± 10%	Maximum Allowable Variation in Subgrade Modulus for ±10% Variation Vertical Displacements.	
	Layered system	Full-Depth System
$E_{asph.\ con.}$	±6.0%	±6.0%
E_{base}	±9.0%	NA
t_{base}	±10.0%	NA
$E_{asph.\ con.} + E_{base} + t_{base}$	±1.0%	NA
$t_{asph.\ con.}$	NA*	±0.6%
$E_{asph.\ con.} + t_{asph.\ con.}$	NA	<±0.0%

Assumed Variations in Subgrade Modulus	Predicted Variation in Vertical Displacement Values	
	Layered System	Full-Depth System
	Asph. Con. and Base Moduli and Base Thickness ±10%	Asph. Con. Modulus and Thickness ±10%
± 5.0%	±11.5%	±16.3%
±10.0%	±14.0%	±20.0%
±20.0%	±19.0%	±27.5%

* No analysis.

STATE OF IDAHO

EXPERIENCE AND APPLICATION

IDAHO FLEXIBLE PAVEMENT DESIGN
USING 5-KIP EQUIVALENT WHEEL LOADING

Idaho used 5000 lb. wheel load as the basic unit loading with a geometric increase in factor for each 1000 lb. increase in wheel load, ie., 5000 = 1, 6000 = 2, 7000 = 4, 8000 = 8, 9000 = 16, 10,000 = 32 etc.

ILLUSTRATION OF COMPUTATION IS GIVEN IN TABLE X FOR 1960.

Figure 1 gives the 5k EWL Trend for various axle groups

Figure 6 shows distribution of axle groups and total 5KEWL/100 vehicles at 47 different stations in Idaho.

Table XI shows vehicle classification counts at 48 stations in Idaho.

Figure 7 shows relationship of total 5KEWL's to 2 axle vehicles as % of total commercial vehicles.

Figure 8 shows relationship of total 5KEWL's to 5 axle vehicles as % of total commercial vehicles.

Figure 9 chart showing relationship of 5KEWL's (1000's) and 2 axle and 5 axle vehicles as % of commercial vehicles. This chart used to establish limits of load classification: heavy, average, light, very light, and residential.

Figure 10 - Traffic index from commercial vehicle count and classification. The traffic indes is computed by the formula:

$$TI = 1.30\ (5KEWL)^{0.12}$$

Figure 11 - Gravel Equivalent Thickness from formula:

$$T = \frac{0.070\ (TI)\ (100-R)}{C^{0.2}}$$

T = Thickness, gravel equivalent ft.

TI = Traffic index

R = Resistance Valve (R_V)

C = Cohesiometer value (normally 20)

Table XIII gives computed degree days (32F) winter precipitation and annual precipitation used to prepare map of different climatic zone factors in Idaho. (Note Idaho factors are multipliers for gravel equivalent thicknesses - do not confuse these multipliers with AASHTO Regional Factors)

Figure 17 - Comparison of Idaho 1957 Design with Idaho 1965 design thicknesses.

Figure 18 - Comparison Idaho 1965 design with AASHTO design for Pt= 2.0.

Figure 19 - Comparison Idaho 1965 design with AASHTO design for Pt = 2.5.

Figure 20 - Comparison of 5KEWL data for 1959 in California and 1960 in Idaho and 1972-80 projected values used by Idaho in design.

Figure 21 - 5KEWL factors used by Idaho various axle groups and illustrated computations for 352 truck.

Idaho Transportation Dept. - Divison of Highways - Materials Section Manual Pavement Design Sections

16-230,231 - Design of flexible pavement

16-232 - Design of asphalt surfacing overlays for asphalt pavements.

TABLE X

EQUIVALENT WHEEL LOAD RATINGS FOR 1960

Distribution of Axle Loading Groups from Statewide Loadometer Survey

		2 Axle Truck			3 Axle Truck			4 Axle Truck			5 Axle Truck			6 Axle Truck		
Axle Gp. Kips	Wh. Load Kips	Wh.Ld. Factor	No. in Group	EWL	Wh.Ld. Factor	No. in Group	EWL	Wh.Ld. Factor	No. in Group	EWL	Wh.Ld. Factor	No. in Group	EWL	Wh. Ld. Factor	No. in Group	EWL
2 - 8	2	.01	2398	24	.01	611	6	.01	294	3	.01	1454	15	.01	42	0
8 - 12	5	1.00	298	298	1.10	291	320	1.10	130	143	1.10	1477	1625	1.20	28	34
12 - 16	7	5.50	162	891	5.90	249	1469	5.80	140	812	6.10	1830	1163	6.30	21	132
16 - 18	8.50	14.00	63	882	15.00	62	930	15.00	43	645	15.00	471	7710	16.00	3	48
18 - 20	9.50	25.00	42	1050	27.00	40	1080	26.00	19	494	27.00	112	3024	28.00	2	56
20 - 22	10.50	41.00	10	410	45.00	27	1215	44.00	4	176	46.00	27	1242	47.00		
22 - 24	11.50	64.00	5	320	69.00	6	414	68.00			71.00	8	568	74.00		
24 - 26	12.50	98.00			106.00			104.00			109.00			113.00		
Total Applications			2978			1286			640			5370			96	
Total EWL				3875			5434			2273			25347			270
EWL/Axle Application			1.301			4.262			3.552			4.720			2.81	
EWL/Year/Vehicle			474			2333			2593			4307			3080	

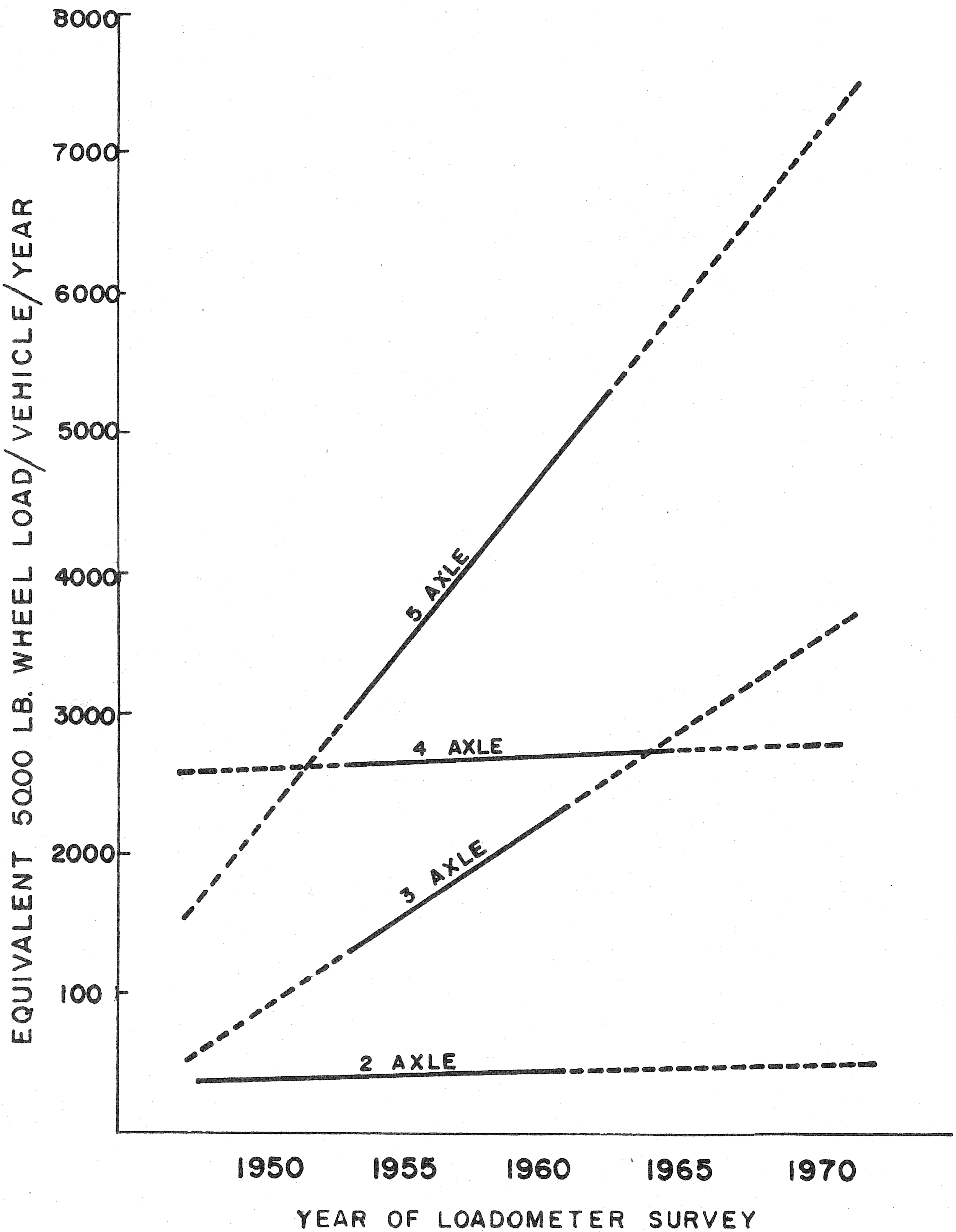


FIGURE I - ALL VEHICLES ——— EWL CONSTANTS

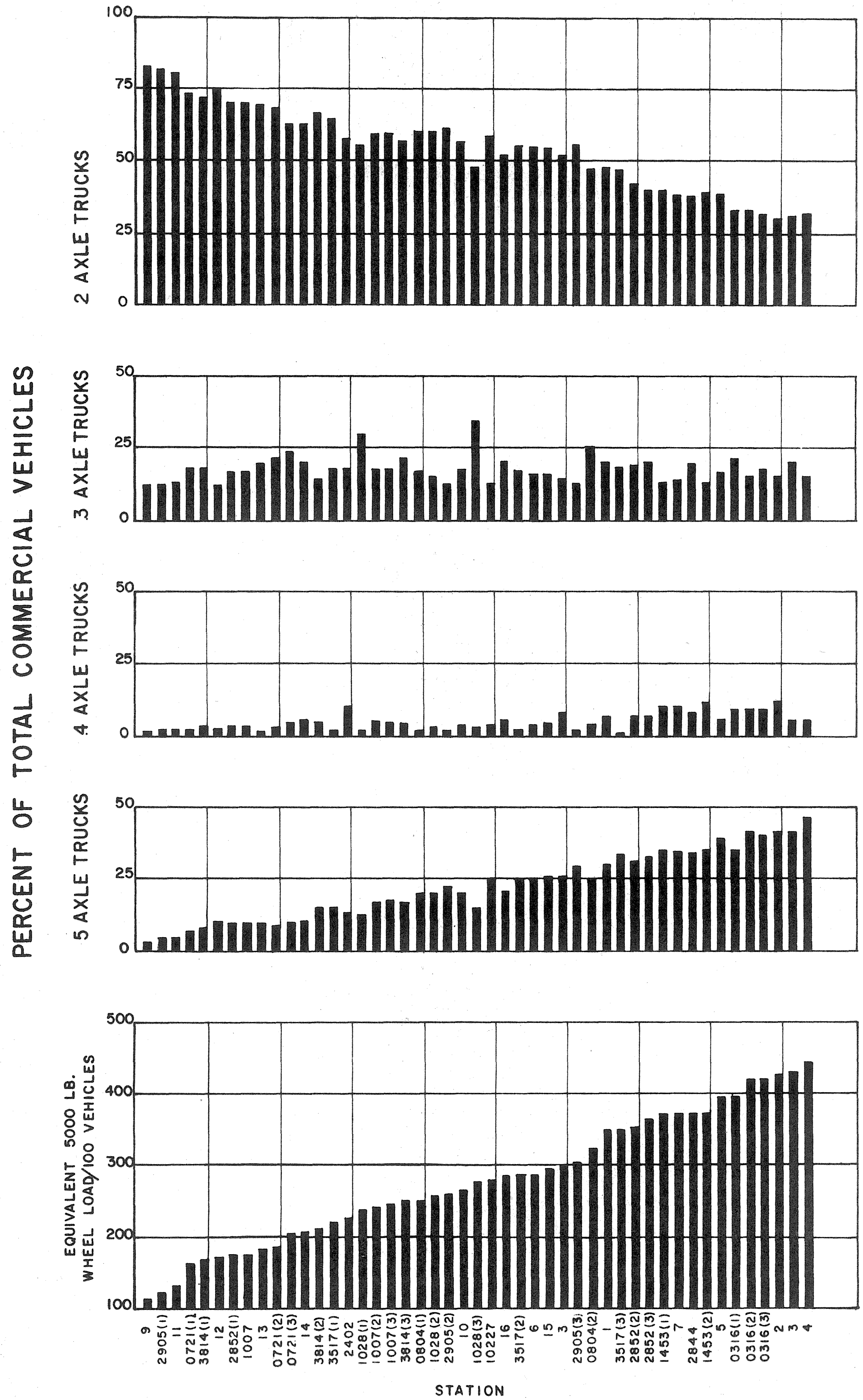


FIGURE 6 — RELATIONSHIPS OF E.W.L. AND COMMERCIAL VEHICLE GROUP PERCENTAGES

TABLE XI

VEHICLE CLASSIFICATION COUNTS

CLASSIFICATION COUNT MADE AT INTERVALS OF 3 MONTHS FOR 3 YEARS DURING PERIOD 1957 - 1961
EACH COUNT IS EQUIVALENT TO ONE 24 HOUR WEEKDAY FOR EACH OF 12 MONTHS

Station No.	Route	Location	ADT 1962 Total	ADT 1962 Trucks Buses	% Trucks Buses	Single Truck 2X	Single Truck 3X	Truck-Semi Trailer 2S-1	Truck-Semi Trailer 2S-2	Truck-Semi Trailer 3S-2	Truck-Trailer Comb. 2S1-2	Truck-Trailer Comb. 2-2	Truck-Trailer Comb. 3-2	Truck-Trailer Comb. 3-3	Buses School	Buses Other
1	US 10	1 Mi. E. Post Falls	8570	760	8.9	42.4	5.7	7.8	3.7	23.3	2.1	1.0	7.6	0.2	1.4	4.9
2	US 30	1 Mi. E. Boise	5455	320	5.9	29.6	4.1	8.7	7.4	30.0	3.7	0.9	12.0	0.1	0.7	2.7
3	US 30	3 Mi. E. Twin Falls	4535	607	13.4	52.0	4.3	4.2	4.1	19.3	3.9	0.4	6.5	0.1	0.9	4.3
4	US 30	8 Mi. S. Pocatello	4535	775	17.1	31.8	3.1	8.5	5.3	32.7	0.5	0.6	13.7	0.3	0.6	2.9
5	US 91	7 Mi. N. Pocatello	3425	560	16.4	38.7	4.0	6.8	4.9	25.7	0.4	0.2	13.2	0.3	1.1	4.8
6	US 95	3 Mi. E. Lewiston	3800	480	12.7	54.9	10.0	4.3	2.6	17.4	0.5	0.3	7.1	0.3	1.5	1.3
7	SH 25	2 Mi. W. Jerome	3510	715	20.4	37.4	3.1	8.0	8.3	28.1	4.0	0.9	7.2	0.1	0.5	2.5
8	US 10	8 Mi. W. Kingston	3550	570	16.0	30.9	9.1	8.0	3.5	33.3	2.3	0.6	8.0	0.4	0.9	3.0
9	SH 19	2.5 Mi. W. Caldwell	2330	235	10.2	82.4	6.6	2.3	1.6	1.6	0.2	0.6	1.4	0.1	2.3	1.0
10	SH 15	3.5 Mi. N. Jct. SH 44	1165	165	14.3	56.9	8.0	5.3	3.3	12.9	0.5	0.4	7.8	0.1	3.1	1.7
11	US 89	1.8 Mi. S. Paris	1135	115	10.0	80.6	3.0	3.3	1.3	4.2	-	0.2	0.7	0.1	6.4	0.2
12	US 26	10 Mi. E. Ririe	1010	160	15.9	74.6	2.7	4.1	2.1	5.8	-	0.2	5.0	0.1	3.6	1.8
13	US 93	2.9 Mi. S. Salmon	800	110	13.6	69.5	7.0	4.6	1.5	6.8	-	0.3	2.2	-	5.3	3.0
14	US 93	5.2 Mi. N. Shoshone	940	135	14.3	62.9	2.0	4.7	5.5	4.9	-	0.6	6.2	-	21.4	8.9
15	US 95	2 Mi. N. Potlatch Jct.	850	120	14.2	53.9	6.0	3.3	3.5	23.7	-	0.1	2.4	-	6.5	0.7
16	US 95	1 Mi. S. Jct. US 2	2475	315	12.7	52.2	10.9	4.4	4.3	18.6	0.5	0.3	3.0	0.1	3.5	2.1
P10227	US 26	Idaho - Wyoming Line	470	80	16.7	58.7	2.1	6.7	3.6	16.7	-	0.2	8.2	-	1.0	2.9
P2844	US 10	Washington - Idaho Line	7100	650	9.2	38.1	5.8	8.5	4.8	25.3	2.2	1.0	8.2	0.3	0.6	5.3
P1453	US 20	5.5 Mi. E. Caldwell - EB	4845	775	16.0	39.1	3.2	7.2	7.6	22.2	3.5	0.6	14.1	0.2	0.5	2.0
P1453	US 20	5.5 Mi. E. Caldwell - WB	4820	780	16.2	40.2	3.2	7.2	7.2	21.5	3.5	0.6	13.7	0.1	0.9	1.9
P2852	SH 41	US 10 - Rathdrum	1335	90	6.7	71.1	7.9	1.3	1.6	7.8	-	1.0	1.2	-	5.6	2.3
P2852	US 10	Coeur d'Alene - Ross Pt.	8100	745	9.2	40.3	6.0	7.5	4.8	25.2	1.8	0.8	7.2	0.2	1.2	5.1
P2852	US 10	Post Falls - Ross Pt.	8570	795	9.3	41.6	5.8	7.3	4.7	24.5	1.7	0.9	7.0	0.2	1.4	4.9
P2905	US 95A	Potlatch Jct. East	2025	260	12.8	61.9	6.2	4.1	2.2	20.6	-	0.1	2.1	0.2	2.5	0.2
P2905	US 95A	Potlatch Jct. South	1530	195	12.7	56.3	6.0	3.6	2.3	26.1	-	0.1	2.1	0.3	3.0	0.2
P2905	SH 6	Potlatch Jct. West	560	80	14.2	82.3	4.9	4.9	1.6	1.6	-	-	2.3	0.4	1.9	0.1
P2402	US 30	Bliss Jct. - Hagerman	950	75	7.9	57.2	3.2	9.3	9.7	10.7	1.0	0.6	2.6	0.1	2.8	2.7
P0721	US 93	Bellevue North	1250	145	11.6	67.4	2.9	6.4	2.5	4.4	0.1	0.7	2.7	0.1	6.4	6.4
P0721	SH 23	Bellevue Southeast	390	65	13.0	73.2	3.3	7.3	1.3	6.3	0.2	0.3	0.7	-	7.2	-
P0721	US 93	Bellevue South	970	115	11.9	63.4	3.2	5.7	2.9	4.8	0.1	0.9	3.7	0.2	5.5	9.7
P0804	SH 15	Horseshoe Bend North	1260	205	16.2	45.4	16.3	5.2	3.3	19.9	0.3	0.1	5.9	0.4	0.3	2.8
P0804	SH 15	Horseshoe Bend South	1080	175	16.3	50.0	16.0	4.9	3.2	15.7	0.4	0.1	5.7	0.1	1.2	2.8
P0804	SH 52	Horseshoe Bend West	400	75	18.2	61.2	12.9	1.5	2.0	16.3	-	-	2.4	0.8	2.4	0.7
P3814	US 95	Ontario Jct. North	5715	395	6.9	71.6	8.8	4.3	1.8	4.9	0.5	0.6	3.2	0.4	1.0	3.0
P3814	US 30	Ontario Jct. South	3455	290	8.3	67.3	4.4	6.3	2.7	9.1	0.8	0.8	4.8	0.4	1.1	2.5
P3814	US 30	Ontario Jct. West	5325	320	6.0	57.0	10.4	6.6	3.0	10.0	0.5	0.3	6.3	0.6	0.7	4.4
P0316	US 30	McCammon - Inkom	3320	610	18.4	32.4	3.1	10.6	7.6	28.2	1.2	0.3	11.3	0.1	1.3	3.0
P0316	US 30	McCammon - Lava	1330	235	17.6	33.5	3.7	16.0	8.3	32.6	0.6	0.4	4.5	-	1.6	0.3
P0316	US 91	McCammon - Arimo	2250	425	18.9	33.1	2.6	7.7	7.1	27.1	1.4	0.3	15.2	0.1	1.1	4.4
P3517	US 95	Spalding Jct. West	3365	450	13.4	55.6	8.8	5.1	1.5	18.1	0.1	0.1	7.1	0.3	1.4	2.0
P3517	US 12	Spalding Jct. East	1350	220	16.4	46.6	11.0	4.0	1.2	27.2	-	0.1	6.3	0.3	2.1	1.4
P3517	US 95	Spalding Jct. South	1950	225	11.6	65.6	6.8	5.5	1.7	6.8	0.1	0.2	7.4	0.3	3.2	2.2
P1028	US 91	Reeds Corner North	2300	415	18.1	59.9	5.0	2.5	2.2	16.3	0.4	0.4	3.9	0.1	1.7	6.7
P1028	US 91	Reeds Corner - Idaho Falls	5000	720	14.4	56.3	4.1	2.8	1.6	9.3	0.5	0.2	2.5	0.1	1.9	20.9
P1028	US 20	Reeds Corner - Arco	2600	395	15.1	48.1	3.3	3.0	1.8	11.8	0.8	0.2	3.0	0.1	1.8	26.4
P1007	US 191	Beach's Corner - Ucon	4615	445	9.6	59.6	7.0	6.9	3.7	12.5	0.6	0.2	4.9	-	2.3	2.2
P1007	US 26	Beach's Corner - Ririe	2000	175	8.9	71.0	6.9	4.6	2.4	5.7	0.1	0.3	3.8	0.1	3.6	1.5
P1007	US 191	Beach's Corner - Idaho Falls	5850	505	8.6	59.9	7.1	7.3	3.6	12.7	0.6	0.2	5.5	0.1	1.0	2.3

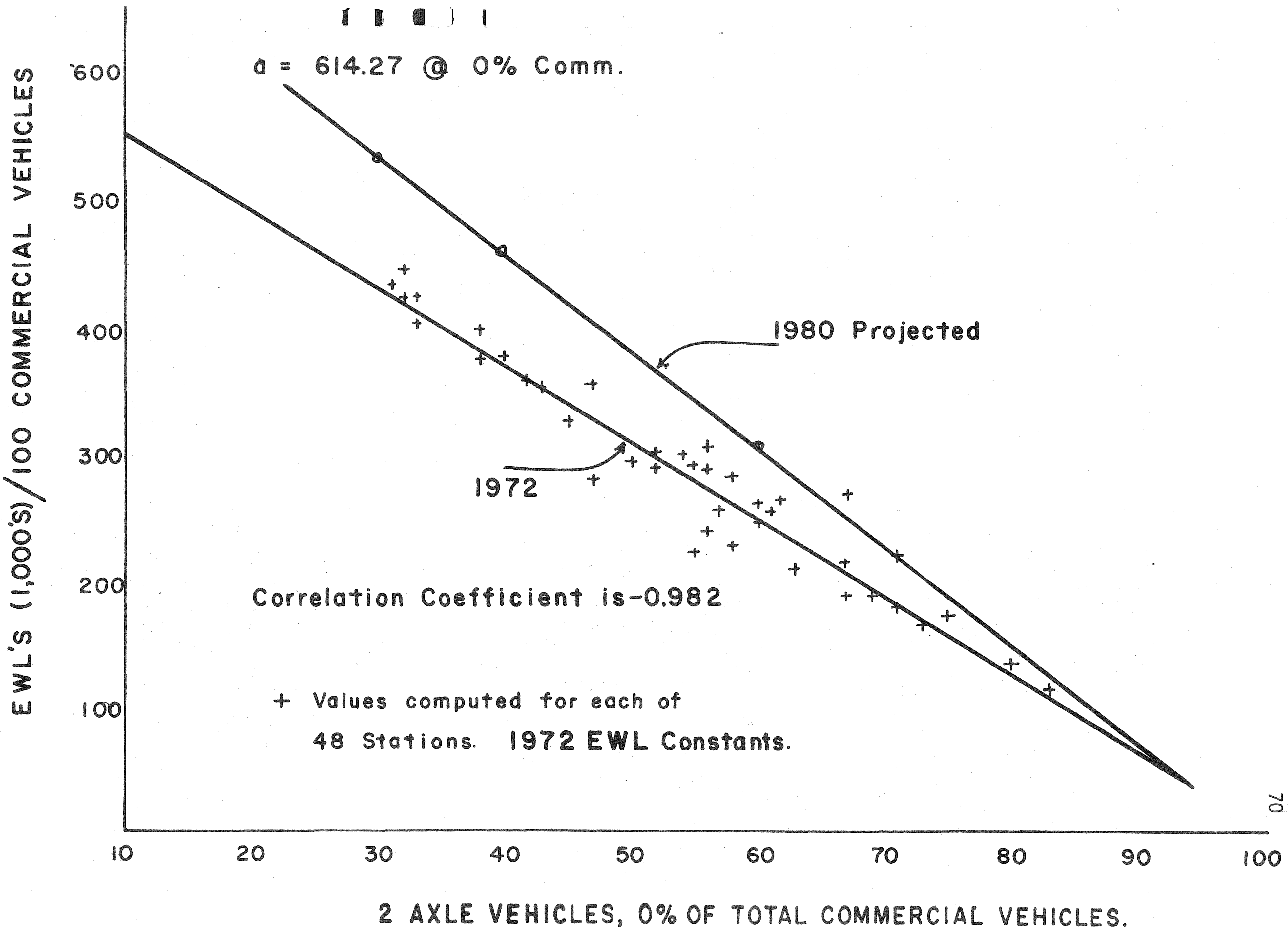


FIGURE 7 — RELATIONSHIP OF TOTAL EWL'S TO 2 AXLE VEHICLES AS % OF TOTAL COMMERCIAL VEHICLES.

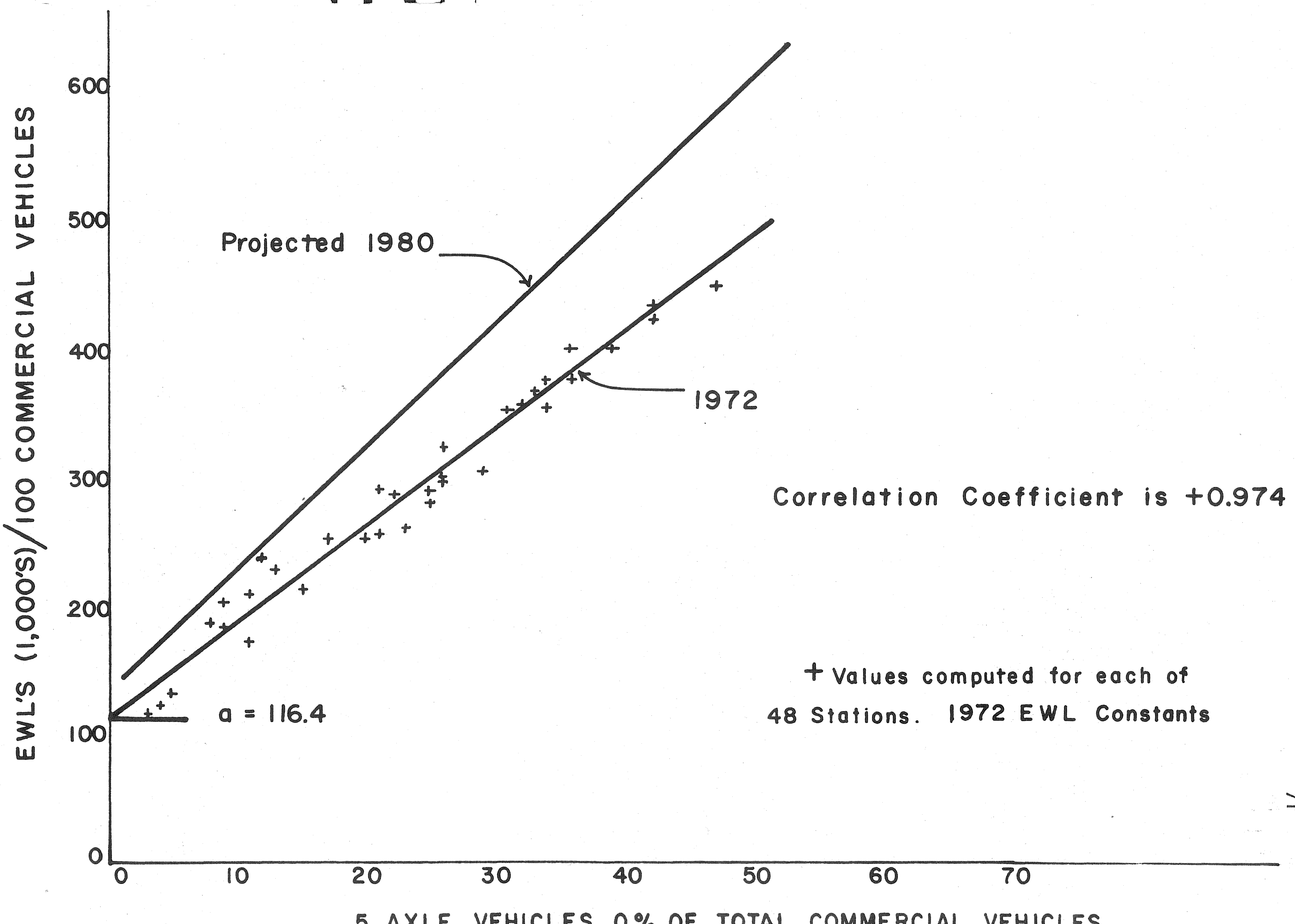


FIGURE 8 - RELATIONSHIP OF TOTAL E.W.L.'S TO 5 AXLE VEHICLES AS % OF TOTAL COMMERCIAL VEHICLES.

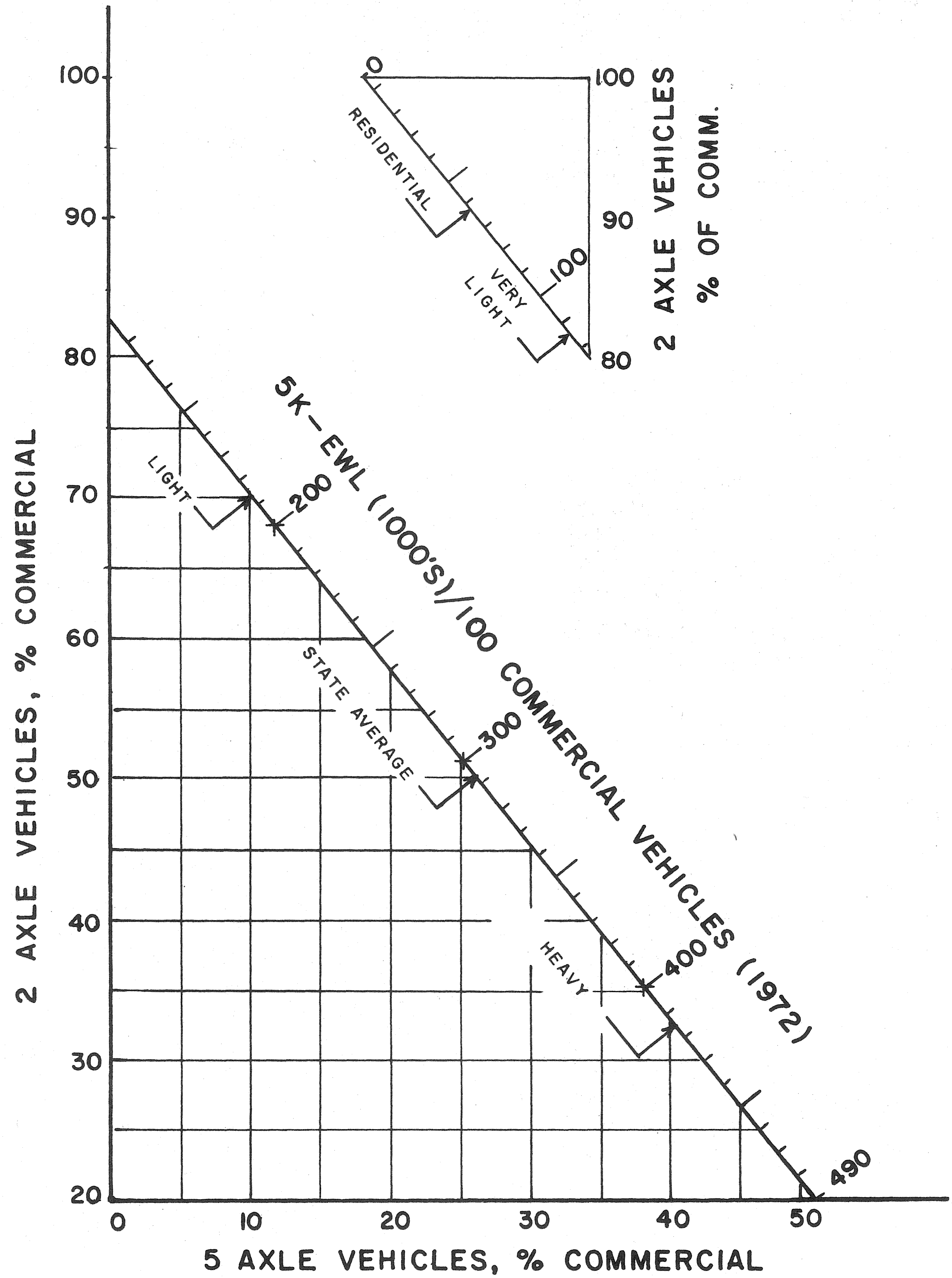


FIGURE 9.- RELATIONSHIP OF 5K-EWL'S (1000'S) AND 2 AXLE AND 5 AXLE VEHICLES AS % OF COMMERCIAL VEHICLES

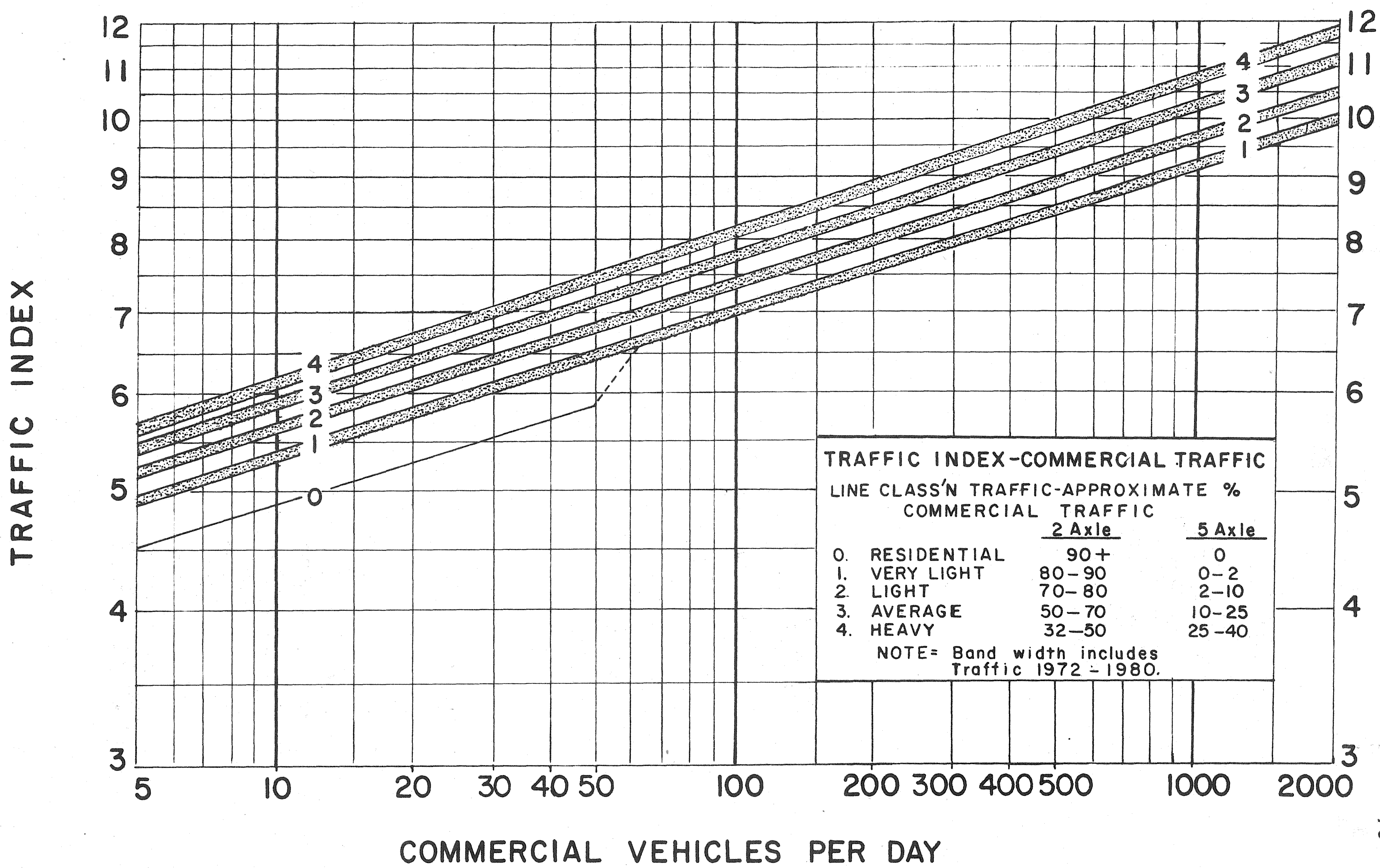


FIGURE 10.-TRAFFIC INDEX FROM COMMERCIAL VEHICLE COUNT & CLASSIFICATION

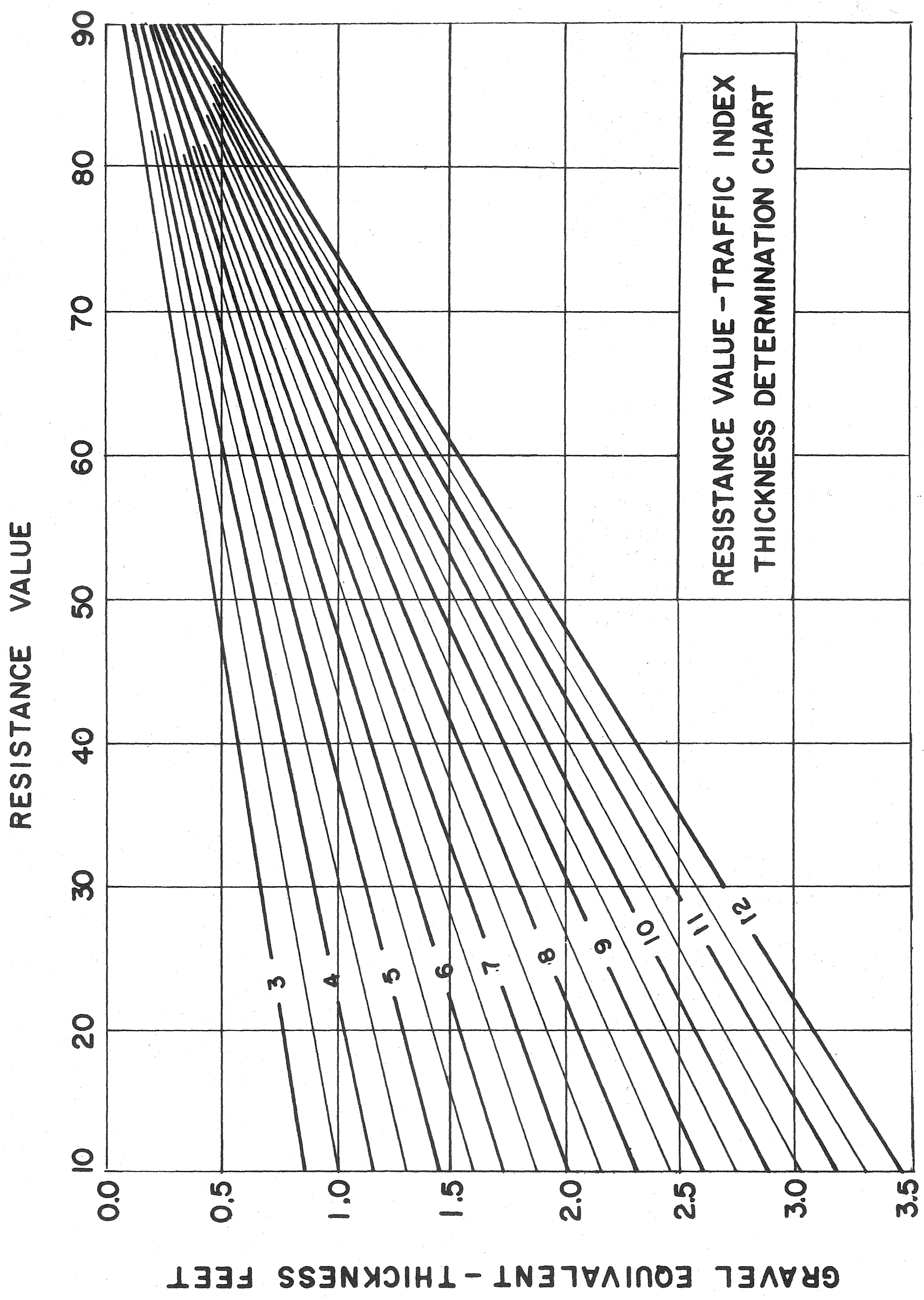


FIGURE 11.— THICKNESS (EQUIVALENT GRAVEL) FROM RESISTANCE VALUE AND TRAFFIC INDEX

TABLE XIII

DEGREE DAYS AND WINTER PRECIPITATION (BELOW 32° F.)

30 Year Mean

	Degree Days 32° F.	Winter Precipitation Inches	Annual Precipitation
Panhandle			
Coeur d'Alene	280	9.0	26.1
Porthill	480	6.3	19.3
Priest River Exp. Sta.	510	13.5	32.9
St. Maries	160	7.5	28.7
Sandpoint	330	13.5	32.7
Northern Canyons			
Kooskia	100	2.8	24.7
Lewiston	100	1.3	13.2
Orofino	100	4.0	25.9
Riggins	-	1.5*	15.5
Northern Prairies			
Grangeville	200	3.5	22.7
Moscow	150	4.2	22.2
Nezperce	220	4.5	20.7
Potlatch	110	6.4	24.5
Central Mountains			
Arrowrock Dam	360	8.0	18.8
Avery R. S.	180	8.3	32.5
Deadwood Dam	1,440	19.0	32.0
Garden Valley	470	10.5	23.8
Hailey R. S.	980	7.6	14.5
Hill City	1,500	9.0	14.7
Idaho City	580	14.0	23.2
Kellogg	200	7.5	31.0
Lowman	670	11.8	24.4
McCall	1,125	14.5	26.8
New Meadows	980	12.0	25.3
Obsidian	1,840	7.7	14.2
Pierce	-	15.0+*	41.3
Wallace	270	15.5	42.1
Wallace - Woodland Park	310	12.5	35.6

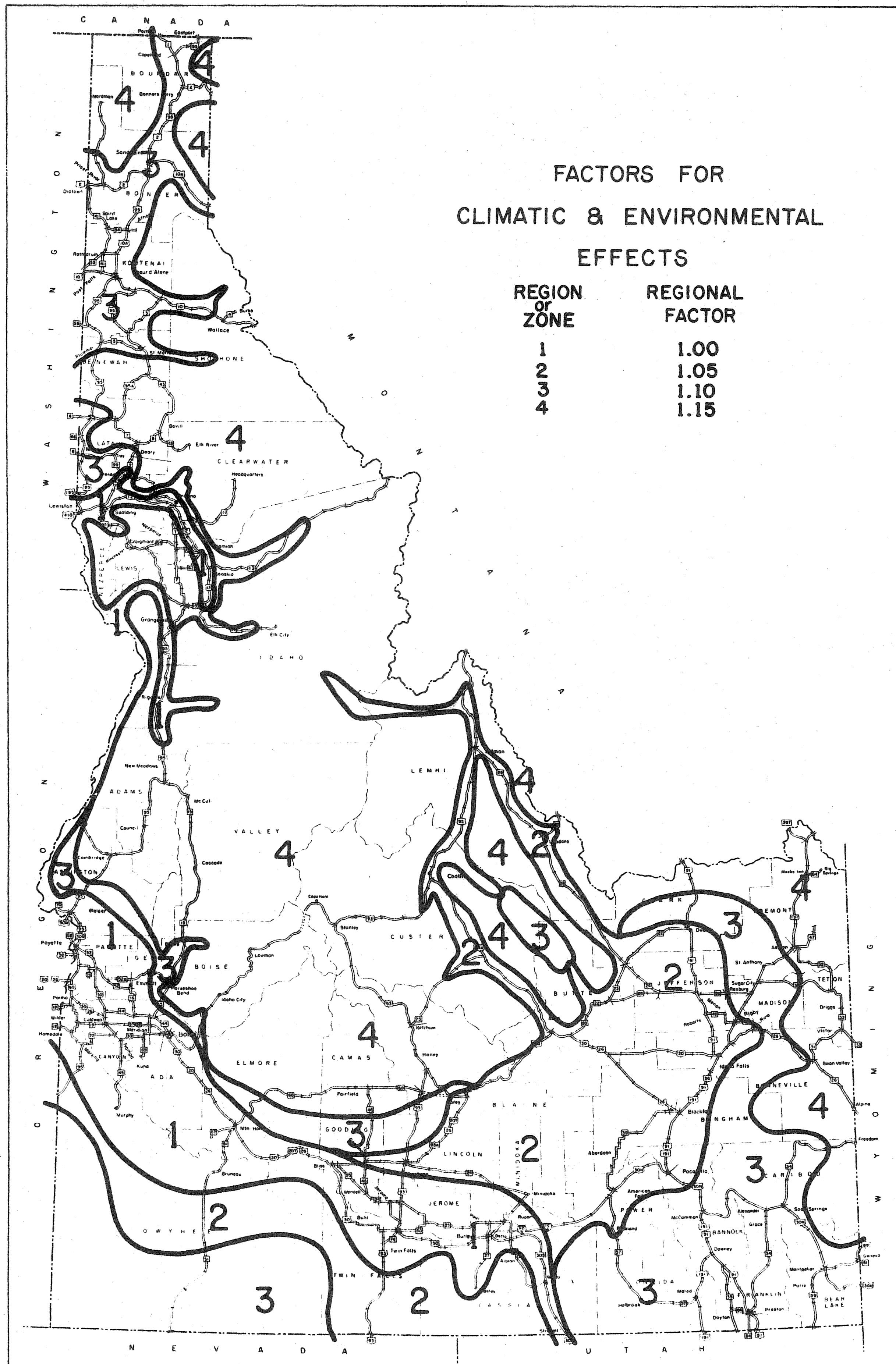
* Estimated

	Degree Days 32° F.	Winter Precipitation Inches	Annual Precipitation
Southwestern Valleys			
Boise A. P.	100	2.0	11.4
Caldwell	100	2.0	10.6
Cambridge	560	9.0	19.7
Council	700	11.5	26.8
Deer Flat Dam	90	1.5	9.3
Emmett	90	3.5	12.3
Glenns Ferry	-	-	8.7
Grandview	0	-	7.3
Kuna	0	-	10.4
Mountain Home	120	1.5	8.8
Parma	125	1.8	9.3
Payette	160	2.5	11.0
Weiser	200	3.0	11.3
Southwestern Highlands			
Hollister	300	2.1	10.0
Central Plains			
Bliss	-	2.2*	8.5
Buhl	200	1.5	8.1
Burley	270	2.0	8.6
Hazelton	200	3.2	10.1
Jerome	300	1.8	8.9
Richfield	-	4.0*	9.6
Rupert	360	2.7	8.3
Shoshone	510	4.0	10.3
Twin Falls	150	1.6	8.7
Northeastern Valleys			
Challis	980	1.5	6.9
Mackay	1,225	3.0	9.3
Salmon City	1,000	2.0	8.9
Upper Snake River Plains			
Aberdeen Exp. Sta.	740	2.3	7.9
Ashton	1,250	8.8	16.9
Blackfoot	-	3.5*	9.9
Dubois Exp. Sta.	850	2.8	10.9
Ft. Hall Indian Agency	570	2.5	9.7

* Estimated

	Degree Days 32° F.	Winter Precipitation Inches	Annual Precipitation
Upper Snake River Plains (Con't)			
Idaho Falls A.P.	900	3.0	8.7
Idaho Falls 42 Mi. N. W.	1,570	2.1	7.0
Idaho Falls 46 Mi. West	1,350	2.8	7.6
Pocatello A.P.	600	3.0	10.8
Sugar City	1,200	4.2	11.2
Eastern Highlands			
Driggs	1,380	5.5	15.8
Grace	1,150	5.2	14.2
Lifton Pump Sta.	1,350	3.0	9.6
Malad	600	4.6	14.0
Montpelier	1,200	5.0	13.7
Oakley	240	1.8	10.1
Preston	650	4.8	15.5

FIGURE 13.


FACTORS FOR
CLIMATIC & ENVIRONMENTAL
EFFECTS
REGION or ZONE | REGIONAL FACTOR
1 | 1.00
2 | 1.05
3 | 1.10
4 | 1.15
CANADA
WASHINGTON
OREGON
NEVADA
UTAH
WYOMING
MONTANA
IDAHO
BOUNDARY
BONNER
KOOTENAI
BENEWAH
SHOSHONE
LATAH
CLEARWATER
NEZ PERCE
LEWIS
ADAMS
VALLEY
LEMHI
CUSTER
BOISE
PAYETTE
CANYON
ADA
ELMORE
CAMAS
BLAINE
BUTTE
JEFFERSON
MADISON
TETON
FREMONT
CLARK
BONNEVILLE
BINGHAM
CARIBOU
BANNOCK
POWER
FRANKLIN
BEAR LAKE
LINCOLN
MINIDOKA
JEROME
TWIN FALLS
CASSIA
OWYHEE
REVISED 6-1-62

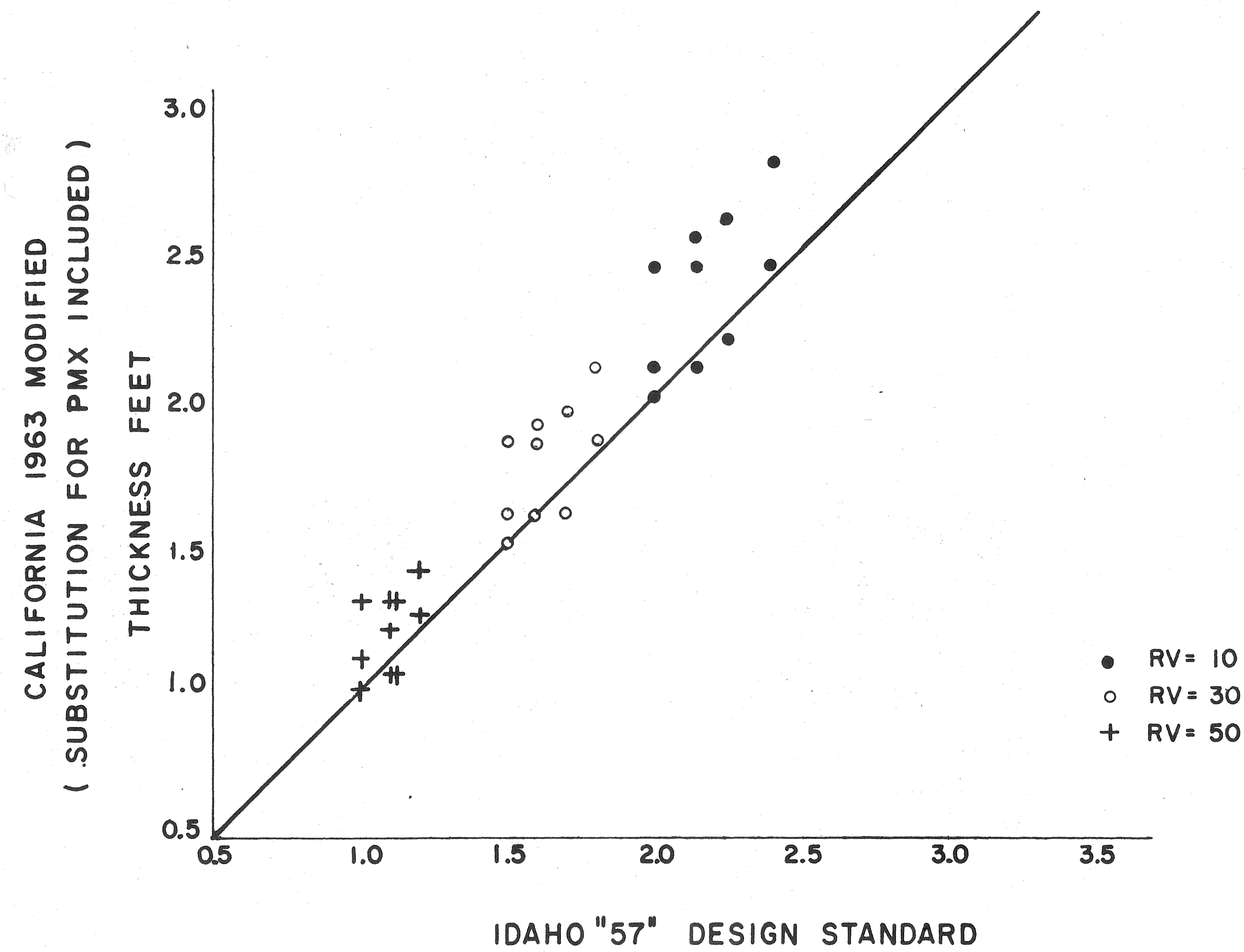


FIGURE 17.— COMPARISON OF DESIGN THICKNESS

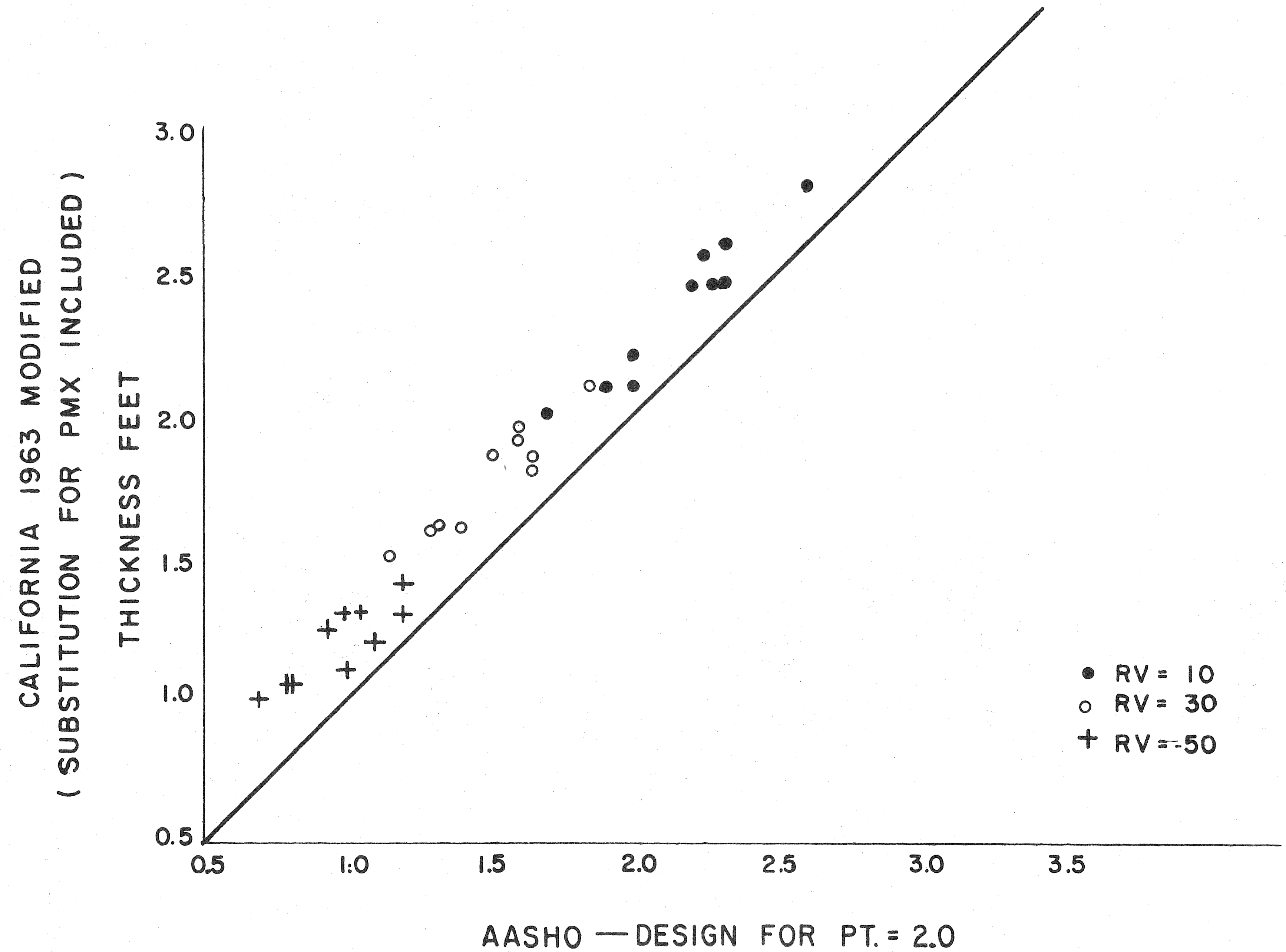


FIGURE 18— COMPARISON OF DESIGN THICKNESS

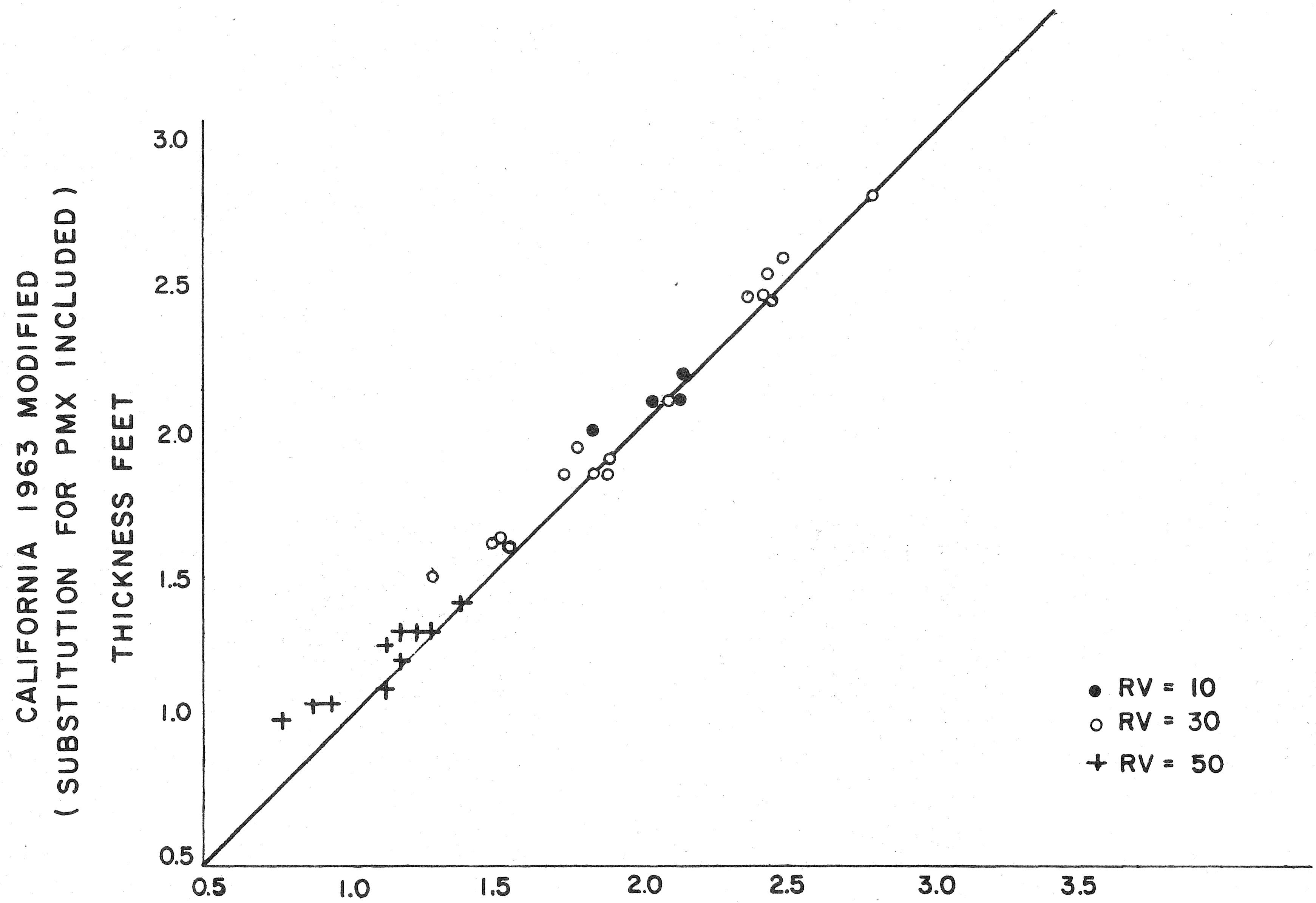


FIGURE 19—COMPARISON OF DESIGN THICKNESSES

FIGURE 20 - COMPARISON OF EWL DATA

No. Axles	California 1959 EWL/ Axle	California 1959 EWL/Yr	Idaho 1960 EWL/ Axle Group*	Idaho 1960 EWL/Yr
2	0.684	499	1.301	474
3	1.489	1630	4.262	2333
4	1.321	1929	3.552	2593
5	2.615	4772	4.720	4307
6	1.349	2954	2.81	3080

* 1 pair tandem axles = 1 group

Projecting 1950 - 1962 data for Idaho gave 1972 -1980 traffic following values:

No. Axles	EWL/Yr
2	510
3	3800
4	2800
5	7600
6	3800 Estimated

FIGURE 21 - EQUIVANCE FACTORS, 5 KIP EWL

AXLE GROUP KIPS	WHEEL LOAD KIPS	WHEEL LOAD FACTOR AXLES PER TRUCKUNIT 2X	3X	4X	5X	6X
2-8	2	0.01	0.01	0.01	0.01	0.01
8-12	5	1.00	1.10	1.10	1.10	1.20
14-16	7	5.50	5.90	5.80	6.10	6.30
16-18	8.5	14.00	15.00	15.00	15.00	16.00
18-20	9.5	25.00	27.00	26.00	27.00	28.00
20-22	10.5	41.00	45.00	44.00	46.00	47.00
22-24	11.5	64.00	69.00	68.00	71.00	74.00
24-26	12.4	98.00	106.00	104.00	109.00	113.00

Note: Tandem axles effect = one single 10% heavier than tandem wheel load
Example: for 352 or 5 axle truck

Axle Load Kips	Wheel Load Factor	No. in Group	EWL
2-8	.01	1471	15
8-12	1.10	1664	1831
12-16	6.10	1707	10413
16-18	15.00	597	8955
18-20	27.00	100	2700
20-22	46.00	9	414
22-24	71.00		
24-26	109.00		

Total Axles	5548
Total EWL	24328
EWL/Axle	4.385
EWL/Vehicle/Yr.	4002

ROADBED STRUCTURE (16-230)

DESIGN FOR FLEXIBLE PAVEMENT (16-231)

16-231.1 Summary of Design Factors.

The major factors to be considered in developing a structural cross-section are:

(1) Structural Quality of the Subgrade Soil: This quality is measured by the stabilometer test as expressed by the Resistance Value (R-value); and by the expansion pressure test.

(2) Traffic: The ADT of total and commercial traffic are used to make projections of traffic for the design period, generally 20 years. Classifications of highways by traffic were developed, based on new computations for 5 kip EWL, making the adjustment in Traffic Index easier. Commercial traffic is classified as "Heavy" for large percentages of 5-axle vehicles and "Light" for large percentages of 2-axle vehicles with "Average" as the third category. Other categories of "Very Light" and "Residential" are used for streets and frontage roads carrying very small volumes of commercial traffic, most of which is 2-axle with only occasional vehicles larger than 3-axle.

(3) Climatic Factors: Climate throughout the State varies from very mild with practically no freezing weather in the lower valleys and low river canyons to the extreme of long, cold winters in the high mountain valleys with several inches of precipitation in the form of snow. The structural cross section is adjusted in thickness in accordance with the severity of climate.

(4) Economic Factors: A satisfactory structural section may be designed using various combinations of materials bearing in mind that economy, both in annual and first cost is, among other factors, an important consideration.

All of the test data necessary to each method of design are reported on a "Soil Evaluation Summary" (Figure 16-231.1) for each profile and borrow soil sample.

16-231.2 Traffic Evaluation.

(1) Scope and Purpose: The magnitude of the load and the number of wheel repetitions are major factors in the performance of a flexible pavement. Owing to the fact that axle-load data are not available for more than a few isolated locations in the State of Idaho, the available data have been combined to give a figure applicable to all routes, with corrections only for the volume and classification of traffic. Classifications of commercial vehicles into Heavy, Average, Light, Very Light, or Residential are made depending on the percentages of

2-AXLE AND 5-AXLE VEHICLES. THE PURPOSE OF OBTAINING THIS INFORMATION IS TO MAKE A REPRESENTATIVE ESTIMATE OF THE 5K-EWL TO BE EXPECTED DURING THE LIFE OF THE PROJECT. ACCORDINGLY, THE ADT USED IN DESIGN HAS BEEN INCREASED FOR ANTICIPATED TRAFFIC, AND THE 5K-EWL ARE EXPRESSED IN TERMS OF THE TRAFFIC INDEX (TI).

(2) EWL CONSTANTS: THE FOLLOWING TABLE LISTS THE CONSTANTS USED TO OBTAIN THE ANNUAL NUMBER OF EQUIVALENT 5,000 LB. WHEEL LOAD REPETITIONS FOR COMMERCIAL VEHICLES:

NUMBER OF AXLES	5K-EWL CONSTANT/YEAR/VEHICLE (FOR 1972-1980)
2	510
3	3800
4	2800
5	7600
6	3800*

*ESTIMATED

THE 5K-EWL FACTOR FOR ANY ROUTE IS THE SUM OF THE 5K-EWL FOR EACH AXLE GROUP AND IS COMPUTED AS FOLLOWS:

AXLE GROUP	DESIGN ADT*	CONSTANT	LIFE, YEARS	5K-EWL
2	76	510	20	775,200
3	20	3800	20	1,520,000
4	8	2800	20	448,000
5	32	7600	20	4,864,000
6	2	3800	20	152,000
				7,759,200

COMM. ADT (PROJECTED) x 5K-EWL CONSTANT x 20 YEARS = 5K-EWL

*TWO DIRECTIONAL COUNT IS DIVIDED BY 2 FOR ONE DIRECTION TRAFFIC.

TRAFFIC INDEX = $1.30\ (EWL)^{0.12}$

TI = $1.30\ (7,759,200)^{0.12} = 8.72$ (USE 8.7.)

FOR CONVENIENCE USE FIGURE 16-231.2 GIVING THE TRAFFIC INDEX FOR VARIOUS TRAFFIC VOLUMES (COMMERCIAL ADT) AND CLASSIFICATIONS OF COMMERCIAL VEHICLES. THESE CLASSIFICATIONS OF VEHICLES ARE AS FOLLOWS:

DH-808 ML 3-67

IDAHO DEPARTMENT OF HIGHWAYS
Central Laboratory
Boise, Idaho

SOILS EVALUATION FOR FLEXIBLE PAVEMENT

Date: Sept. 3, 1968
Project No: I-80N-3(15)176   280-315
Section US-93 - SH-50
County JEROME
Auth. No. 1022-2213

NOTE: 4 Lanes, 38-ft. Roadway width, P crown 0.02 crown slopes.

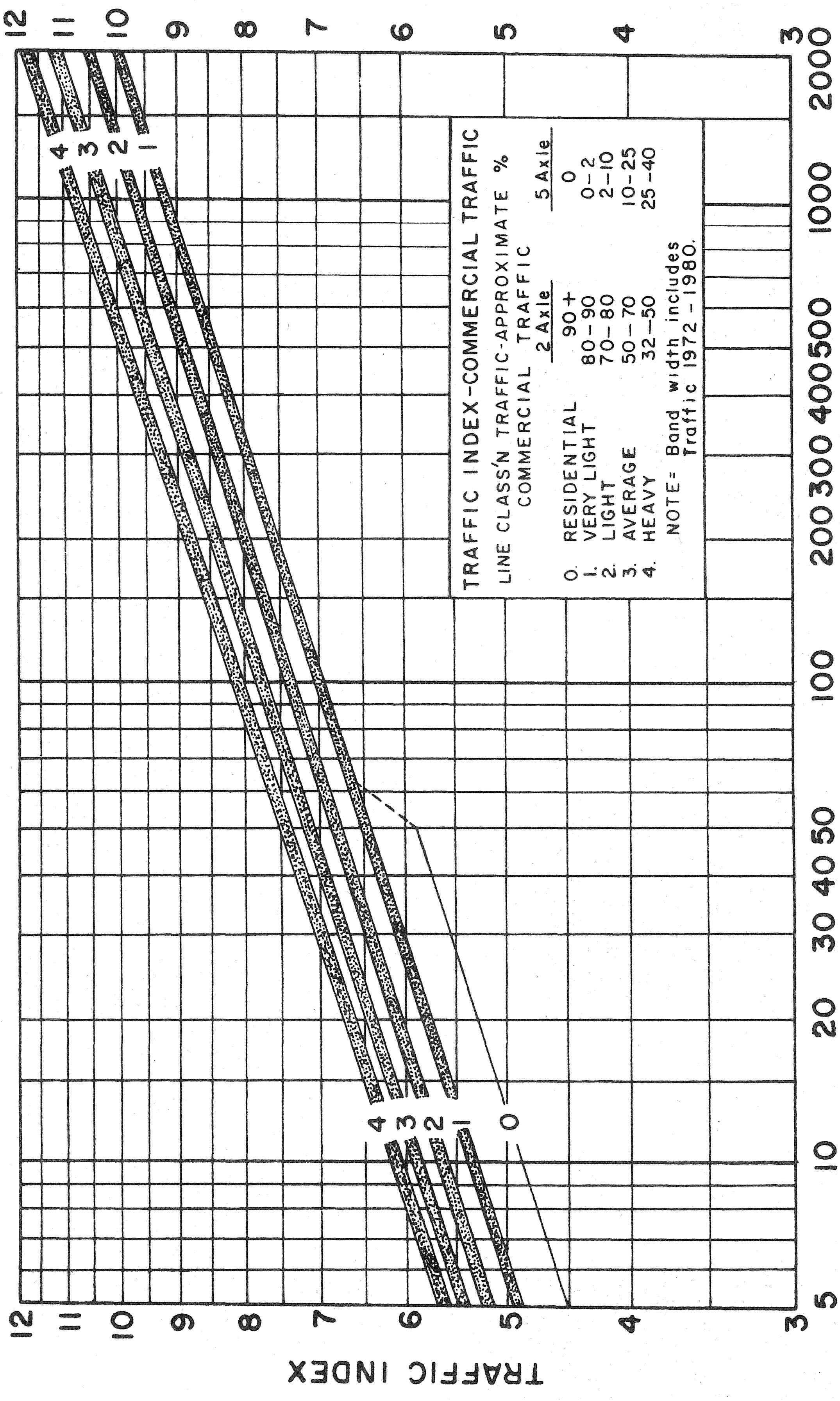
TRAFFIC DATA	1958	19_
ADT	7032	503_
Comm'l ADT	244	60_
Ave.Comm'l ADT	424 ÷2=	212

EWL Class'n Heavy Traffic Index_
Climatic Factor 1.0

Station	Lab. No.	Depth, ft.	Unified Soil Class'n	L.L.	P.I.	FME	L.S.	#10	#40	#200	wt./C.F.	Opt. Moist.	"R" Value	Expansion Pressure, p. s. i.	Recommended Thick., Ft. "R"	Recommended Thick., Ft. Exp.	Blanket
625+00	183955	0-10	ML	22	2	23.1	2.0	94	91	63	105.1	16.5	70	1.36	0.9	1.6	N
P.t. Jr-4	189687	0-10	ML	22	4	20	3.2	93	84	64	116.1	13.2	35	1.11	1.9	1.3	N

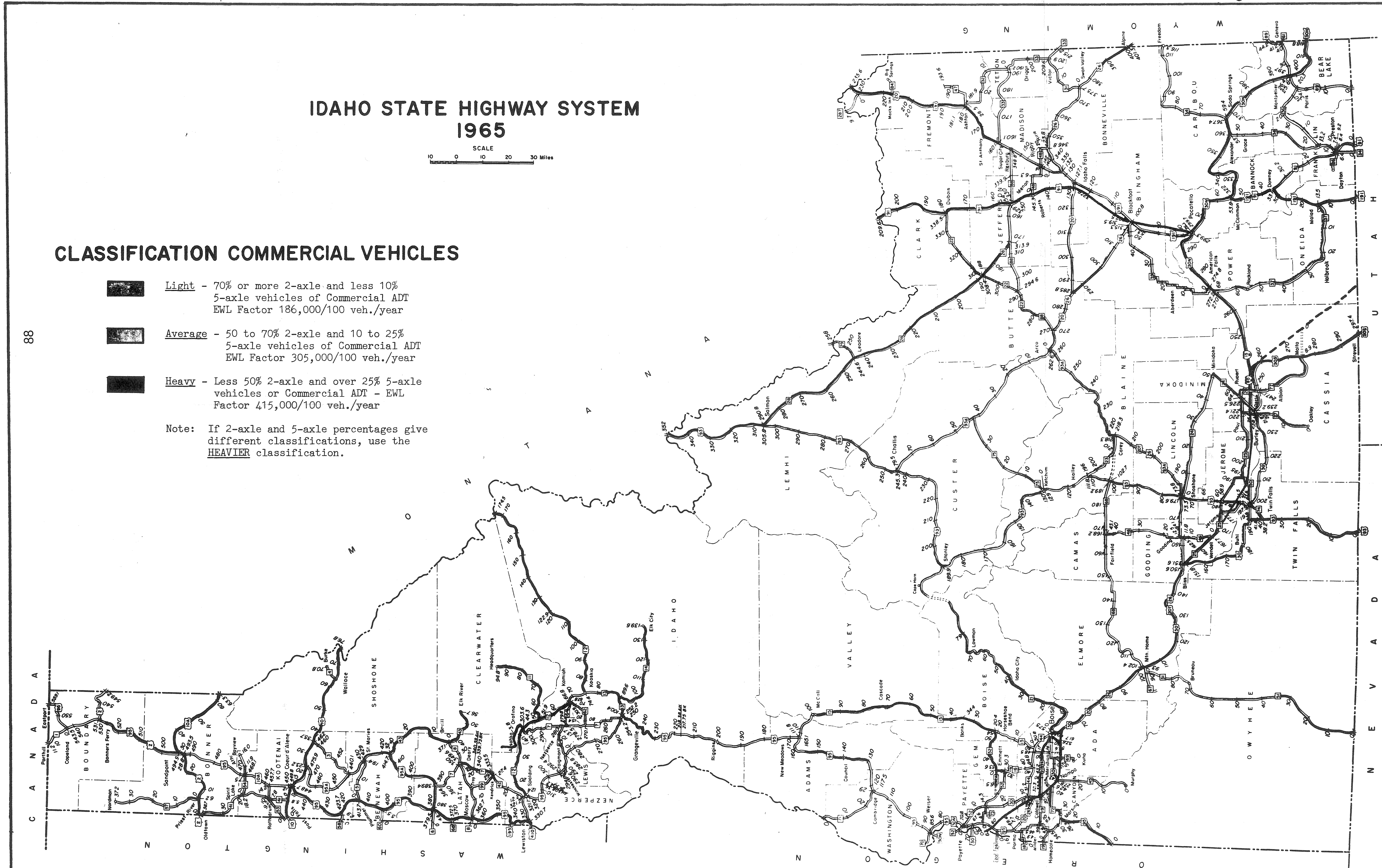
*Wt./C.F. taken from "R" Value Determination.
+Wt./C.F. corrected for +3/4-in. material.
NOTE: The recommended thicknesses are expressed in terms of untreated aggregate base corrected for climatic factor.

_____ P. E.
Materials Engineer



COMMERCIAL VEHICLES PER DAY

TRAFFIC INDEX FROM COMMERCIAL VEHICLE COUNT & CLASSIFICATION


IDAHO STATE HIGHWAY SYSTEM
1965
SCALE
10 0 10 20 30 Miles
CLASSIFICATION COMMERCIAL VEHICLES
Light - 70% or more 2-axle and less 10% 5-axle vehicles of Commercial ADT EWL Factor 186,000/100 veh./year
Average - 50 to 70% 2-axle and 10 to 25% 5-axle vehicles of Commercial ADT EWL Factor 305,000/100 veh./year
Heavy - Less 50% 2-axle and over 25% 5-axle vehicles or Commercial ADT - EWL Factor 415,000/100 veh./year
Note: If 2-axle and 5-axle percentages give different classifications, use the HEAVIER classification.
CANADA
MONTANA
WYOMING
UTAH
NEVADA
OREGON
WASHINGTON

CLASSIFICATION	PER CENT OF COMMERCIAL TRAFFIC 2-AXLE	5-AXLE
HEAVY	32-50	25-40
AVERAGE	50-70	10-25
LIGHT	70-80	2-10
VERY LIGHT	80-90	0-2
RESIDENTIAL	90+	0

IF THE TOTAL COMMERCIAL COUNT FOR RESIDENTIAL CLASSIFICATION EXCEEDS 50 VEHICLES PER DAY, USE THE "LIGHT" CLASSIFICATION. INTERSTATE PROJECTS SHALL ALWAYS BE CLASSIFIED AS "HEAVY." IF THE CLASSIFICATION FROM 2-AXLE AND 5-AXLE DIFFERS, USE THE HEAVIER CLASSIFICATION FOR DESIGN.

COMMERCIAL VEHICLE COUNTS SHALL BE COMPUTED AS FOLLOWS:

2-LANE HIGHWAYS - ONE-HALF OF THE AVERAGE OF THE ADT FOR THE BEGINNING AND THE END OF THE DESIGN LIFE PERIOD. (NORMALLY 20 YEARS.)

4-LANE HIGHWAYS - SAME AS FOR 2-LANE WITH 100 PER CENT COMMERCIAL TRAFFIC ASSIGNED TO OUTER LANE.

FRONTAGE ROADS - SAME AS FOR 2-LANE.

INTERCHANGE RAMPS - 100 PER CENT OF AVERAGE ASSIGNED ADT FOR DESIGN LIFE.

THE STATE HIGHWAY SYSTEM SHOULD NOT BE DESIGNED FOR A TRAFFIC INDEX OF LESS THAN SIX. ABNORMAL DISTRIBUTION OF EXTREMELY HEAVY VEHICLES, I.E., LOGGING OR MINING TRAFFIC CAN RESULT IN A TRAFFIC INDEX GREATER THAN THAT ASSIGNED AS "HEAVY." SHOULD THIS BE SUSPECTED, THE TRAFFIC INDEX SHALL BE COMPUTED AS SHOWN IN SECTION 16-231.2.

TRAFFIC DATA FOR EACH PROJECT ARE PRESENTED IN THE PROJECT DESIGN BROCHURE. INCLUDED ARE: TOTAL ADT, PER CENT COMMERCIAL VEHICLES. CLASSIFICATION OF THE COMMERCIAL TRAFFIC AS "HEAVY," "AVERAGE," OR "LIGHT" CAN BE DETERMINED FOR THE INTERSTATE AND PRIMARY ROUTES IN FIGURE 16-231.21. FOR THE SECONDARY ROUTES MODIFY THE DATA FROM FIGURE 16-231.21 WITH AVAILABLE LOCAL KNOWLEDGE OF THAT ROUTE.

ALSO INCLUDED ARE TRAFFIC DIAGRAMS FOR INTERSECTIONS AND INTERCHANGE RAMPS. SUCH A DIAGRAM IS ILLUSTRATED IN FIGURE 2-431.21 OF THE SURVEYS & PLANS MANUAL. THIS DIAGRAM ILLUSTRATES ONE-WAY TRAFFIC MOVEMENTS. THE ADT FOR ANY SEGMENT IS DETERMINED BY ADDING ALL TURNING MOVEMENTS WHICH AFFECT THAT SEGMENT.

SINCE THE FIGURE FOR PER CENT TRUCKS WAS DEVELOPED FOR THE TRAVELED WAY, IT MUST BE APPLIED TO THE RAMPS WITH DISCRETION. EACH TURNING MOVEMENT MUST BE EXAMINED CAREFULLY TO DETERMINE THE INFLUENCE OF LOCAL CONDITIONS ON THE POTENTIAL VOLUME OF COMMERCIAL TRAFFIC.

16-231.3 Design by R-Value

(1) The Resistance (R) Value is a test value which measures the ability of a soil to resist lateral flow due to vertically applied loads. The test is conducted using the Hveem Stabilometer (See Idaho T-8) wherein the soil is tested at an applied load of 4000 lb. The R-Values obtained by testing a soil at four conditions of moisture are plotted as shown in Figure 16-231.31. The intersection of this curve with the 4000-lb. ordinate gives the design R-Value.

(2) The formula for total flexible pavement thickness follows:

$$T = \frac{0.\cancel{070}\;{}^{.0058}\,(TI)\,(100-R)}{C^{0.2}}$$

T = Thickness in Feet

TI = Traffic Index

R = Resistance Value

C = Cohesiometer Value (Normally taken as 20)

For convenience Figure 16-231.3 has been provided for the solution of this formula. The value of C is taken as 20 for untreated crushed aggregates.

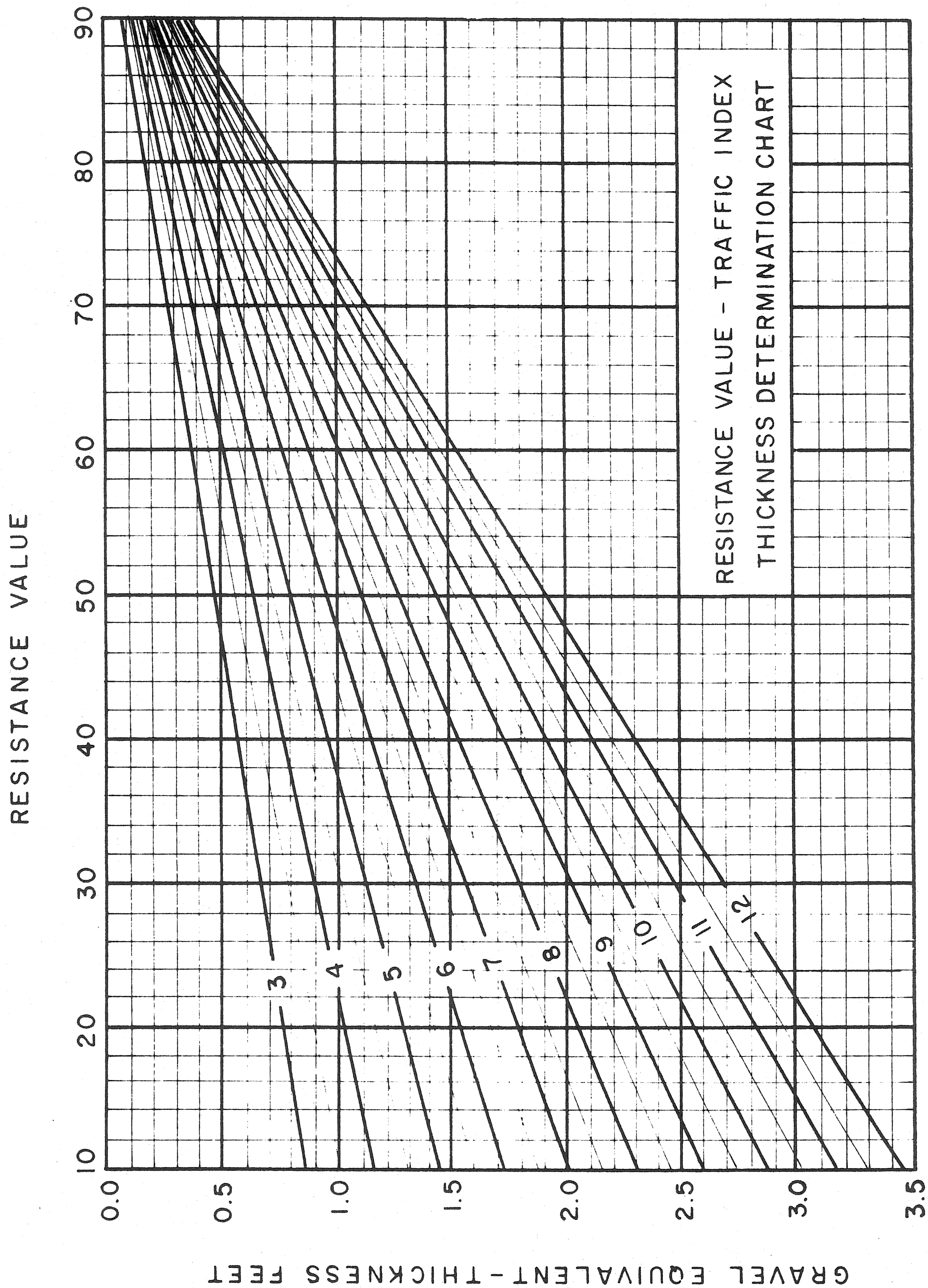
See Highway Research Board Record No. 13, "Thickness of Flexible Pavements by the California Formula Compared to AASHO Road Test Data" by F.N.Hveem and G.B.Sherman.

16-231.4 Design by Expansion Pressure

Given expansion pressure data from Idaho T-8, "Standard Method of Compaction of Soils and Soil Mixtures for the Expansion Pressure and Hveem Stabilometer Tests," a curve is plotted as shown in Figure 16-231.31. The design expansion pressure is obtained where this curve intersects the diagonal balance line. The balance line represents the condition at which the ballast requirements from R-Value at the governing Traffic Index are equal to those from expansion pressure. The overlying material must provide sufficient weight to prevent any volume change in the subgrade soil caused by expansion. The unit weight of this material is assumed to be 130 lb. per cubic foot for most granular materials with the exception of some volcanic aggregates. The thickness in feet necessary to confine soil with a given expansion pressure is:

$$B = \frac{\text{Exp. Pressure (psi)}}{\text{Unit Wt. Agg., Lb./Cu.Ft.}} \times 144$$

The design thickness may be read directly from the curves, Figure 16-231.41. Structural elements of flexible pavement are illustrated in Figure 16-231.4



THICKNESS (EQUIVALENT GRAVEL) FROM
RESISTANCE VALUE AND TRAFFIC INDEX

DH-803 4-67

IDAHO DEPARTMENT OF HIGHWAYS
Central Laboratory
Boise, Idaho

Distribution:
Hwy. Engr.
Dist.Engr.
Res. Engr.

Lab. No. 183955

Report of Tests on SOIL

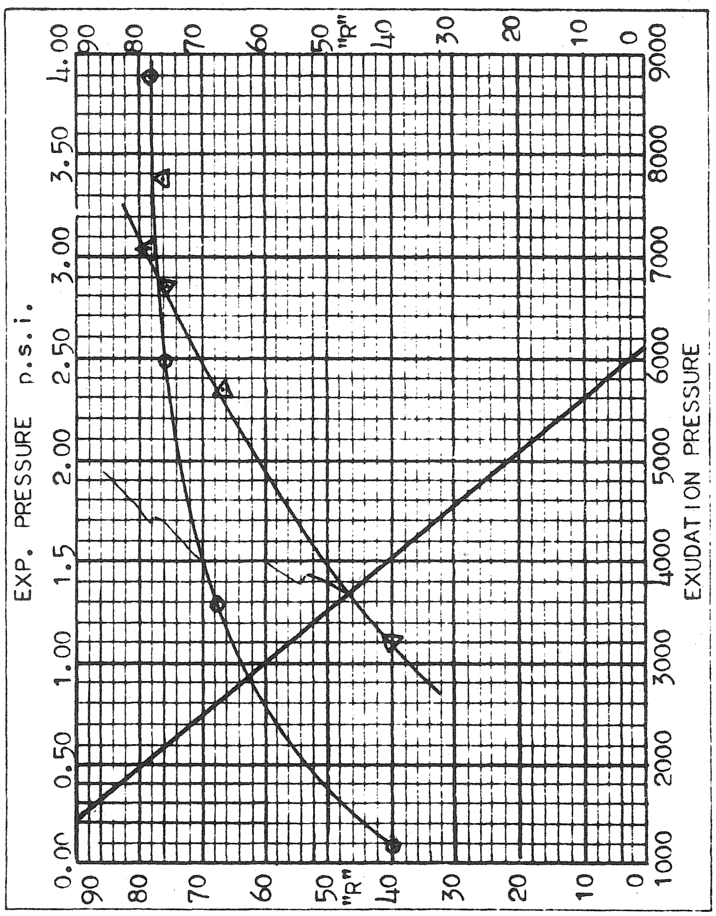
Project I-15-3(15)176 280-315 County Jerome Source No.
Submitted by S. LARSON Ident. No. SL/12622-2213/3P
Station 625+00 Layer No. 1 Depth 0.0-1.0'
Description of Soil Silt Date Sampled 6/24/68 Received 6/26/68

Mechanical Anal. % Pass		Soil Constants	
3" Sq.	100	Liquid Limit	22
2" Sq.	98	Plastic Limit	20
1" Sq.	97	Plasticity Index	2
3/4" Sq.	96	Field Moist.Equiv.	23.1
1/2" Sq.	96	Linear Shrinkage	2.1
No. 4	95	Specific Gravity(+3/4")	2.62
No. 10	94	Specific Gravity(-No.4)	
No. 20	93	Sand Equivalent	
No. 30	92	"R" Value	70
No. 40	91	Exp. Pressure, psi	1.36
No. 50	89	Unified Class'n	ML
No. 100	78	AASHO Class'n	A-4(6)
No. 200	66	Traffic Index	9.0

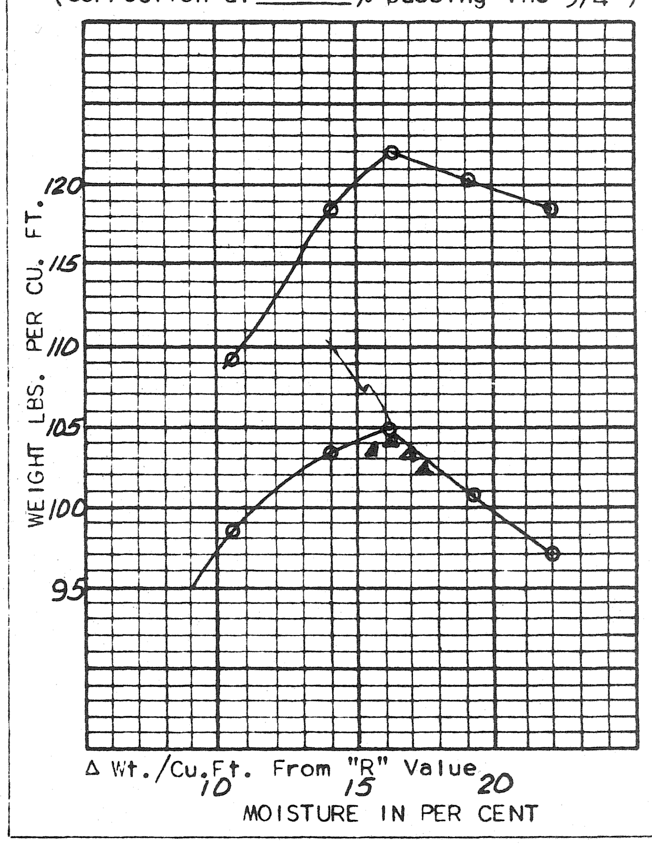
CMP Data

pH- Resistivity ohm. Cm.
Est. Time To Perforation (16 ga.)
Add 12 years for Bituminous Coating

Remarks



MOISTURE-DENSITY CURVE
AASHO DESIGNATION T-99 METHOD A
Max.Dry Wt. 105.1 #/Cu.Ft. Opt.Moist. 16.5 %
Corrected Max. Dry Wt.= lb/Cu.Ft.
(Correction at % passing the 3/4")

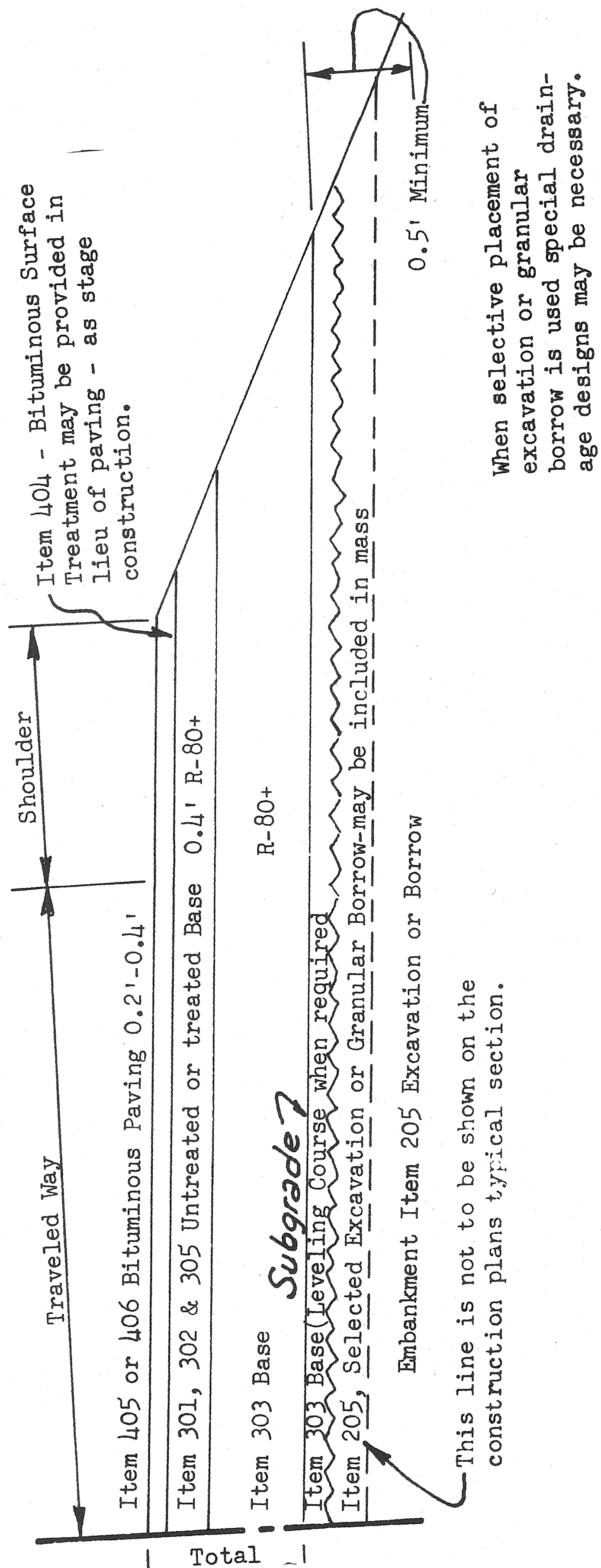


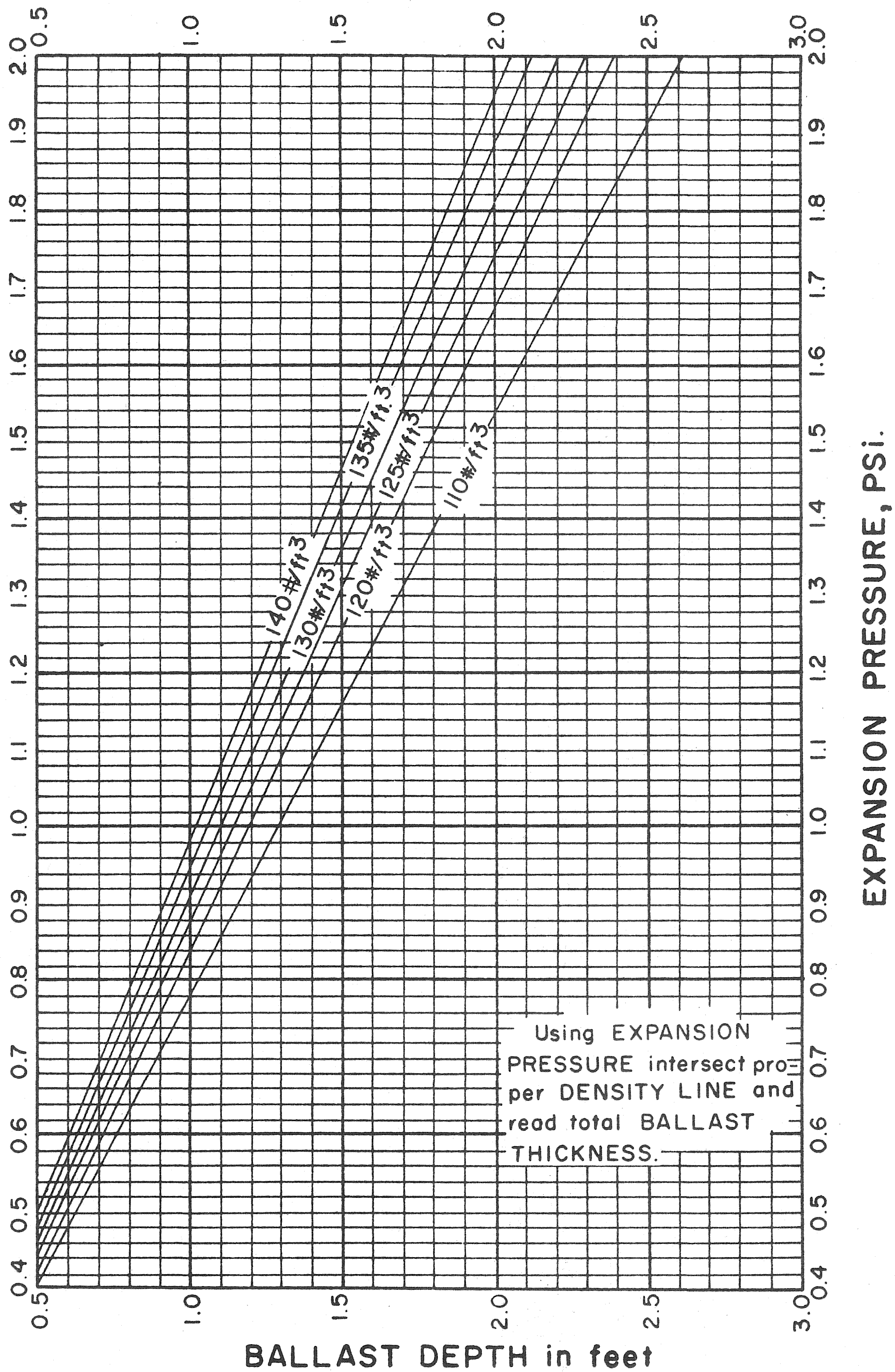
This report covers only material as represented by this sample and does not necessarily cover all soil from this layer or source.

Date Mailed 7/3/68

P. E.
Materials Engineer

STRUCTURAL ELEMENTS OF FLEXIBLE PAVEMENTS




EXPANSION PRESSURE, PSi.
BALLAST DEPTH in feet
0.4
0.5
0.6
0.7
0.8
0.9
1.0
1.1
1.2
1.3
1.4
1.5
1.6
1.7
1.8
1.9
2.0
0.5
1.0
1.5
2.0
2.5
3.0
140#/ft3
135#/ft3
130#/ft3
125#/ft3
120#/ft3
110#/ft3
Using EXPANSION PRESSURE intersect proper DENSITY LINE and read total BALLAST THICKNESS.

16-231.5 DESIGN ADJUSTMENT FOR CLIMATIC FACTOR

THE CLIMATIC FACTOR PROVIDES FOR A MEASURE OF ADDITIONAL PROTECTION AGAINST WINTER AND SPRING WEATHER. THIS FACTOR PROVIDES ADDED THICKNESS AS FOLLOWS:

DESIGN THICKNESS = THICKNESS FROM R-VALUE X F (CLIMATIC FACTOR)

F = 1.00 FOR REGION 1
F = 1.05 FOR REGION 2
F = 1.10 FOR REGION 3
F = 1.15 FOR REGION 4

THE VARIOUS REGIONS WERE DELINEATED THROUGH STUDY OF PRECIPITATION RECORDS DURING THE PERIODS WHEN THE 30-YEAR MEAN TEMPERATURE REMAINED BELOW 32 DEGREES F., AND FROM THE EXPERIENCE OF DISTRICT MAINTENANCE DIVISIONS. FIGURE 16-231.5 SHOWS REGIONAL AREAS TO BE USED.

16-231.6 DESIGN ADJUSTMENT FOR COHESIVE MATERIALS

THE COHESION OF COMPACTED ASPHALT MIXTURES OR CEMENT TREATED MIXTURES GIVES ADDITIONAL STRENGTH TO THE PAVEMENT STRUCTURE. IT IS POSSIBLE TO ADJUST THE TOTAL THICKNESS IN CONSIDERATION OF THIS COHESIVE STRENGTH FROM THE THICKNESS AS DETERMINED FROM R-VALUE DESIGN ADJUSTED FOR CLIMATIC EFFECTS. THIS REDUCTION IN TOTAL PAVEMENT STRUCTURE THICKNESS IS OBTAINED THROUGH USE OF SUBSTITUTION RATIOS.

TABLE 1

SUBSTITUTION RATIOS FOR SURFACING AND TREATED BASE
FOR AGGREGATE BASE MATERIALS - (GRAN.BS: SURF./OR TR BS)

TRAFFIC INDEX	PLANT MIX PAVEMENT (HOT)	PLANT MIX BASE (HOT)	ROAD MIX PAVEMENT (COLD) CEMENT TREAT BASE ROAD MIX BASE (COLD)	GRANULAR BORROW *
OVER 7.0	2.0:1	1.75:1	1.50:1	0.75:1
5.5-6.9	2.5:1	2.00:1	1.75:1	0.75:1
LESS 5.4	3.0:1	2.5: 1	2.00:1	1.00:1

*MAY INCLUDE CINDER AGGREGATE AND SELECTED GRANULAR EXCAVATION IF QUALITY IS ADEQUATE.

NOTE: DESIGN THICKNESSES SHALL NOT BE LESS THAN 0.5 FT. ACTUAL DEPTH FOR RESIDENTIAL STREETS AND COUNTY SECONDARIES AND 0.8 FT. ACTUAL DEPTH FOR STATE HIGHWAY PROJECTS.

16-231.7 EXAMPLE OF DESIGN

TO ILLUSTRATE FLEXIBLE PAVEMENT DESIGN, ASSUME THE FOLLOWING THREE EXAMPLES:

EXAMPLE 1:

	1963 DATA	1983 PROJECTION
TOTAL ADT	2032	5030
COMMERCIAL ADT (FIGURE 16-231.1)	244	603
CLASS 2-AXLE = 40%, 5-AXLE = 30% OF COMM. (FIGURE 16-231.21)	HEAVY	HEAVY
R-VALUE OF SUBGRADE SOIL	70	
EXPANSION PRESSURE OF SUBGRADE SOIL	1.36 PSI	
WT./C.F. BASE AND SURFACING MATERIAL, LB.	135	
CLIMATIC REGION	1	
DESIGN STANDARD	INTERSTATE HIGHWAY, 4-LANE DIVIDED, 0.4 FT. SURFACE COURSE AND 0.4 FT. TREATED BASE.	

THIS IS A 4-LANE DIVIDED INTERSTATE PROJECT; TRAFFIC VOLUMES ARE ASSUMED TO BE DIVIDED EQUALLY IN BOTH DIRECTIONS AND WITH VIRTUALLY ALL COMMERCIAL TRAFFIC USING THE OUTER LANES. THEREFORE, EACH SIDE OF THE INTERSTATE IS TO BE DESIGNED FOR ONE-HALF OF THE TRAFFIC VOLUME. THE AVERAGE COMMERCIAL TRAFFIC VOLUME FOR THE 20-YEAR DESIGN LIFE IS $\frac{244 + 603}{2} = 424$ ADT

AND WITH HALF ON EACH SIDE, THIS GIVES 212 ADT FOR DESIGN PURPOSES.

USING FIGURE 16-231.2 AND THE COMMERCIAL TRAFFIC VOLUME OF 212 AND CLASSIFIED HEAVY, THE TRAFFIC INDEX IS 9.0. USING FIGURE 16-231.3 A TRAFFIC INDEX OF 9.0 AND AN R-VALUE OF 70 GIVES A TOTAL UNADJUSTED THICKNESS (GRAVEL EQUIVALENT) OF 0.9 FT.

USING FIGURE 16-231.41 WITH A 1.36 PSI EXPANSION PRESSURE AND THE 135 LB./C.F. FOR BASE AND SURFACING MATERIALS RESULTS IN A REQUIRED TOTAL THICKNESS OF 1.6 FT. THEREFORE EXPANSION PRESSURE THICKNESS GOVERNS.

THE CLIMATIC FACTOR FOR REGION 1 IS 1.0 RESULTING IN NO INCREASE FOR CLIMATE. (FIGURE 16-231.5).

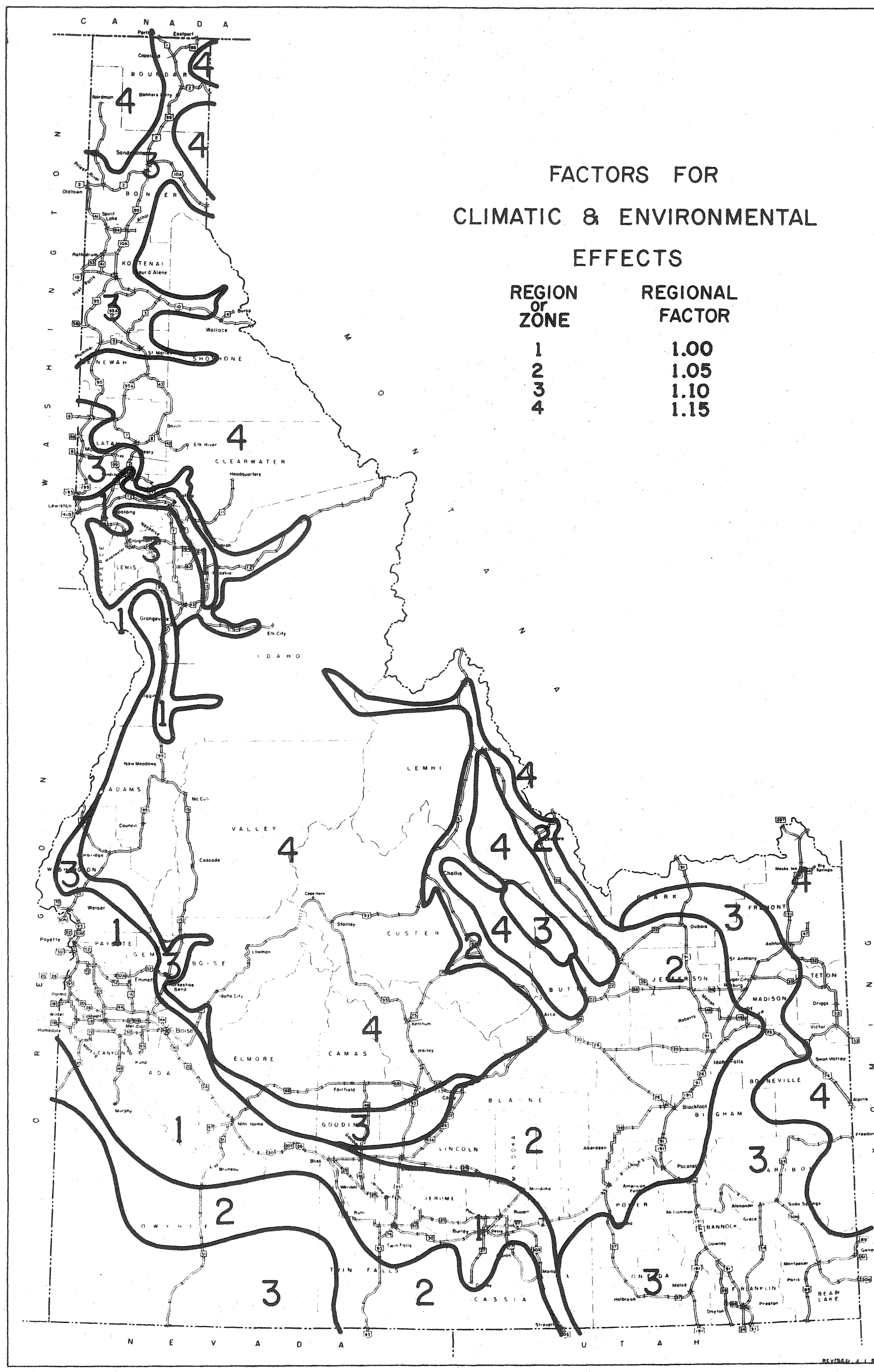
SINCE THE DESIGN STANDARD PROVIDES FOR 0.4 FT. OF PLANT MIX AND 0.4 FT. OF TREATED BASE, THE THICKNESS (GRAVEL EQUIVALENT THICKNESS) WILL BE ADJUSTED FOR THE COHESIVE STRENGTH OF THESE MATERIALS USING THE RATIOS FROM TABLE 1.

THEY ARE AS FOLLOWS:

0.4 FT. PLANT MIX = 0.4 x 2 = 0.8 FT. OF AGGREGATE BASE;
0.4 FT. PLANT MIX ASPHALT TREATED BASE = 0.4 x 1.75 = 0.7 FT. OF AGGREGATE BASE.

ASSUMING THAT PLANT MIX ASPHALT TREATED BASE WILL BE USED, THEN THE DESIGN THICKNESS FOR UNTREATED BASE BECOMES:

0.9 FT. (GR.EQ.) - 0.8 FT. (PLANT MIX) - 0.7 FT. (ATB) =-0.6 FT.


FACTORS FOR
CLIMATIC & ENVIRONMENTAL
EFFECTS
REGION or ZONE
REGIONAL FACTOR
1 1.00
2 1.05
3 1.10
4 1.15
CANADA
NEVADA
UTAH
IDAHO
LEMHI
VALLEY
CUSTER
ELMORE
CAMAS
BLAINE
LINCOLN
CASSIA
CLEARWATER
ADAMS
LEWIS

THEREFORE, NO UNTREATED BASE IS REQUIRED BY R VALUE OF THE SUBGRADE SOIL. NEXT DETERMINE THICKNESS NEEDED TO SATISFY THE ACTUAL BALLAST DEPTH REQUIREMENT OF EXPANSION PRESSURE.

1.6 FT. (GR.EQ.) - 0.4 FT. (PLANT MIX) - 0.4 FT. (ATB) = 0.8 FT.

USE 0.8 FT. OF UNTREATED BASE.

THE TYPICAL SECTION WOULD THEN SHOW

0.4 FT. PLANT MIX PAVEMENT
0.4 FT. PLANT MIX ASPHALT TREATED BASE
0.8 FT. UNTREATED AGGREGATE BASE COURSE

THIS SECTION PROVIDES AN ACTUAL TOTAL THICKNESS OF 1.6 FT. AND A GRAVEL EQUIVALENT TOTAL THICKNESS OF 2.3 FT.

EXAMPLE 2:

	1963 DATA	1983 PROJECTION
TOTAL ADT	5000	8000
COMMERCIAL ADT	600	900
CLASS 2-AXLE = 40%, 5-AXLE = 30% OF COMM.	HEAVY	HEAVY
R-VALUE OF SUBGRADE SOIL	30	
EXPANSION PRESSURE, PSI	0.61	
WT., LB./C.F., CEMENT TREATED BASE AND PLANTMIX	135	
CLIMATIC REGION	2	
DESIGN STANDARD	INTERSTATE HIGHWAY 4-LANE DIVIDED, 0.4 FT. SURFACE COURSE AND 0.4 FT. TREATED BASE.	

THIS IS A 4-LANE DIVIDED INTERSTATE PROJECT. EACH SIDE OF THE INTERSTATE IS TO BE DESIGNED FOR ONE-HALF OF THE TRAFFIC VOLUME. THE AVERAGE COMMERCIAL TRAFFIC VOLUME FOR THE 20-YEAR DESIGN LIFE IS $\frac{600 + 900}{2} = 750$ ADT

AND WITH HALF ON EACH SIDE THIS GIVES 375 ADT FOR DESIGN PURPOSES, COMMERCIAL TRAFFIC CLASSIFIED AS HEAVY.

USING FIGURE 16-231.2 AND THE COMMERCIAL TRAFFIC VOLUME OF 375, CLASSIFIED HEAVY THE TRAFFIC INDEX IS 9.4. EXPANSION PRESSURE REQUIRES A THICKNESS OF 0.7 FT. FROM FIGURE 16-231.41. A TRAFFIC INDEX OF 9.4 AND AN R-VALUE OF 30 GIVES A TOTAL UNADJUSTED THICKNESS (GRAVEL EQUIVALENT) OF 2.15 FT. THE CLIMATIC FACTOR FOR REGION 2 IS 1.05. MULTIPLYING THE GRAVEL EQUIVALENT THICKNESS BY THE FACTOR 2.15 x 1.05 = 2.25 FT., GIVES THE DESIGN REQUIREMENT UNADJUSTED FOR SURFACING OR BASE COURSES.

SINCE THE DESIGN STANDARD PROVIDES FOR 0.4 FT. OF PLANT MIX AND 0.4 FT. OF TREATED BASE, THE THICKNESS (GRAVEL EQUIVALENT THICKNESS) WILL BE ADJUSTED FOR THE COHESIVE STRENGTH OF THESE MATERIALS USING THE FACTOR FROM TABLE 1 ASSUMING THE AGGREGATE IS MORE COMPATIBLE WITH CEMENT AND A CEMENT TREATED BASE IS MORE ECONOMICAL. THEY ARE AS

0.4 FT. PLANT MIX = 0.4 x 2.0 = 0.8 FT. OF GRANULAR BASE
0.4 FT. CEMENT TREATED BASE = 0.4 x 1.50 = 0.6 FT. AGGREGATE BASE

ASSUMING THAT THE AGGREGATE IS MORE COMPATIBLE WITH PORTLAND CEMENT THE DESIGN THICKNESS FOR UNTREATED BASE THEN BECOMES:

2.25 FT. (GR.EQ.) - 0.8 FT. (SURF.) - 0.6 FT. (CTB) = 0.85 FT. USE 0.9.

THE TYPICAL SECTION WOULD THEN SHOW:

0.4 FT. PLANT MIX SURFACING
0.4 FT. CEMENT TREATED BASE
0.9 FT. UNTREATED AGGREGATE BASE COURSE
1.7 FT. TOTAL THICKNESS

CHECKING THE DESIGN FOR EXPANSION PRESSURE OF THE SUBGRADE SOIL THE 0.61 PSI EXPANSION PRESSURE AND THE 135 LB/C.F. FOR BASE AND SURFACING MATERIALS RESULTS IN A TOTAL THICKNESS REQUIREMENT OF 0.70 FT. HAD EXPANSION PRESSURES RESULTED IN A THICKNESS GREATER THAN 1.70 FT., THE THICKNESS FROM EXPANSION PRESSURE WOULD GOVERN THE TOTAL THICKNESS REQUIREMENT.

EXAMPLE 3:

THIS IS A RAMP ON THE INTERSTATE PROJECT AS SHOWN IN EXAMPLE 1.

THIS EXAMPLE IS PRESENTED TO ILLUSTRATE THAT IT IS NECESSARY TO ESTABLISH A NEW EXPANSION PRESSURE BALANCE LINE WHEN THE TRAFFIC INDEX CHANGES FOR A GIVEN SOIL.

	1968 DATA	1988 PROJECTION
TOTAL ADT	125	292
COMMERCIAL ADT	15	35
CLASS (FIG. 16-231.21)	HEAVY	HEAVY
R-VALUE OF SUBGRADE SOILS	70	
CLIMATIC REGION		
DESIGN STANDARD	INTERSTATE HIGHWAY 2-LANE RAMP, 0.3 FT. PLANT MIX PAVEMENT AND 0.3 FT. TREATED BASE.	

THE PER CENT COMMERCIAL VEHICLES (%C) OF THE ADT IS 12%.

THE TRAFFIC VOLUME ON THE RAMP IS IN ONE DIRECTION AND THEREFORE IS NOT DIVIDED BY 2 AS IN EXAMPLE 1 AND 2. THE AVERAGE COMMERCIAL TRAFFIC VOLUME FOR THE 20-YEAR DESIGN LIFE IS $\frac{35 + 15}{2} = 25$ ADT. CLASS OF COMMERCIAL TRAFFIC IS "HEAVY".

USING FIGURE 16-231.2 AND THE COMMERCIAL TRAFFIC VOLUME OF 25, THE TRAFFIC INDEX IS 7.0.

Using Figure 16-231.3, a traffic index of 7.0 and R-Value of 70, this gives a total unadjusted thickness (gravel equivalent) of 0.7 Ft.

The expansion pressure balance line is determined from Figure 16-231.71 and with the use of the test report (Figure 16-231.31). This expansion pressure value for the changed traffic condition is selected at the intersection of expansion pressure curve and the expansion pressure balance line. See Figure 16-231.7. This gives an expansion pressure of subgrade soil equal to 1.14 psi.

Using Figure 16-231.41 with the 1.14 psi expansion pressure and 135 lb./C.F. for weight of base and pavement materials results in a required thickness of 1.2 Ft.

The climatic factor for Region 1 is 1.0 (Figure 16-231.5) resulting in no increase for climate.

It should be noted at this point that if a comparison is made with Example 1, the change in traffic index from a 9.0 to 7.0 has decreased the required thickness for the ramps from 0.9 to 0.7 ft. gravel equivalent by R-Value and from 1.6 Ft. to 1.2 Ft. actual thickness by expansion pressure.

Since the minimum design standard requires 0.3 ft. of plant mix and 0.3 ft. of treated base, the gravel equivalent thickness for the cohesive strength of these materials using the ratios from Table 1, section 16-231.6 are as follows:

0.3 Ft. Plant Mix = 0.3 x 2 = 0.6 Ft. Aggregate Base.
0.3 Ft. Plant Mix Asphalt Treated Base = 0.3 x 1.75 = 0.53 Ft. Aggregate Base.

Assuming that plant mix treated base will be used, then the design thickness for untreated base becomes:

0.7 Ft. (GrEq.) - 0.6 Ft. (Plant Mix) - 0.5 Ft.(Plant Mix Base) = -0.4 Ft.

Therefore the minimum design standard for ramps more than satisfies the requirement of the subgrade soils as required by R-Value.

Now to check the minimum design standard against the actual ballast depth requirement of expansion pressure.

1.2 Ft. - 0.3 Ft. Plant Mix - 0.3 Ft. (Plant Mix Base) = 0.6 Ft.

Use 0.6 Ft. of untreated base.

The typical section would then be:

0.3 Ft. Plant Mix Pavement
0.3 Ft. Plant Mix Treated Base
0.6 Ft. Untreated Aggregate Base

This typical section provides an actual total thickness of 1.2 Ft. and a gravel equivalent total thickness of 1.7 Ft. Note that this provides an over design by R-Value criteria but just satisfies expansion pressure criteria of the subgrade soil.

16-231.8 DESIGN BY DEFLECTIONS

PAVEMENT THICKNESS DESIGN BASED ON BENELMAN BEAM DEFLECTIONS MAY BE UTILIZED, PROVIDED THAT THERE IS A SUFFICIENT NUMBER TAKEN AT APPROPRIATE TIMES OF THE YEAR AND PROPERLY CORRELATED WITH OTHER PROJECTS OR DESIGN PROCEDURES AS POSSIBLE. THIS DESIGN PROCEDURE IS TO BE IMPLEMENTED THROUGH THE HEADQUARTERS MATERIALS SECTION.

THESE DESIGNS BY DEFLECTION ARE PARTICULARLY APPROPRIATE FOR MAINTENANCE BETTERMENT AND RECONSTRUCTION PROJECTS.

DH-803 4-67

Distribution:
Hwy. Engr.
Dist.Engr.
Res. Engr.

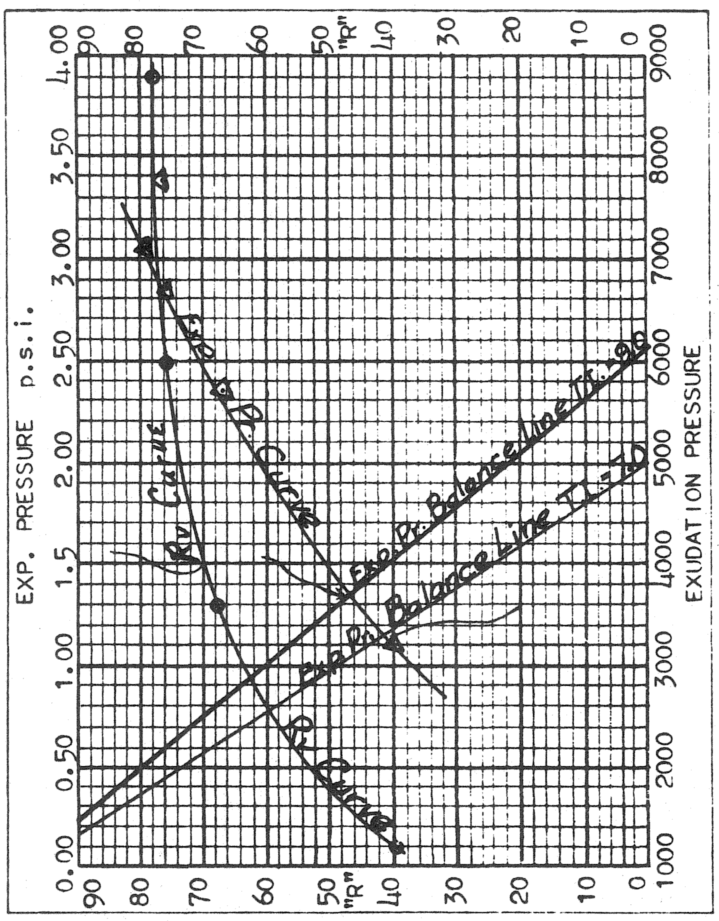
IDAHO DEPARTMENT OF HIGHWAYS
Central Laboratory
Boise, Idaho

Lab. No. 183955

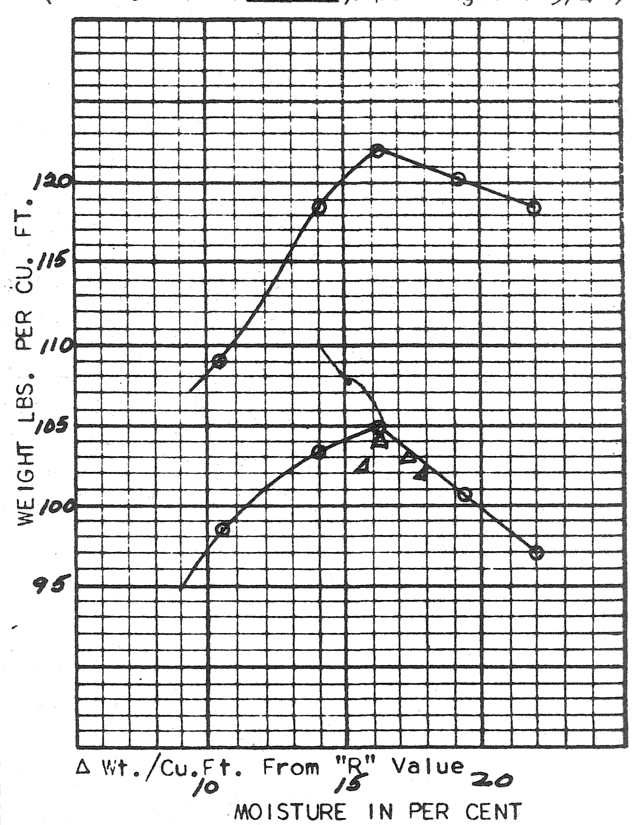
Report of Tests on SOIL

Project I-15-3(15) 176 280-315 County Jerome Source No. ______
Submitted by S. Larson Ident. No. SL/12022-2213/3P
Station 625-000 Layer No. 1 Depth 0.0 - 1.0'
Description of Soil Silt Date Sampled 6/24/68 Received 6/26/68

Mechanical Anal. % Pass		Soil Constants		CMP Data
3" Sq.	100	Liquid Limit	22	pH- ______ Resistivity ______ ohm. Cm.
2" Sq.	98	Plastic Limit	20	Est. Time To Perforation (16 ga.) ______
1" Sq.	97	Plasticity Index	2	Add 12 years for Bituminous Coating
3/4" Sq.	96	Field Moist.Equiv.	23.1	
1/2" Sq.	96	Linear Shrinkage	2.1	Remarks
No. 4	95	Specific Gravity(+3/4")	2.62	
No. 10	94	Specific Gravity(-No.4)		
No. 20	93	Sand Equivalent		
No. 30	92	"R" Value	70	
No. 40	91	Exp. Pressure, psi	1.14	
No. 50	89	Unified Class'n	ML	
No. 100	78	AASHO Class'n	A-4(6)	
No. 200	66	Traffic Index	7.0	



MOISTURE-DENSITY CURVE
AASHO DESIGNATION T-99 METHOD A
Max.Dry Wt. 105.1 #/Cu.Ft. Opt.Moist. 16.5 %
Corrected Max. Dry Wt.= ______ lb/Cu.Ft.
(Correction at ______ % passing the 3/4")

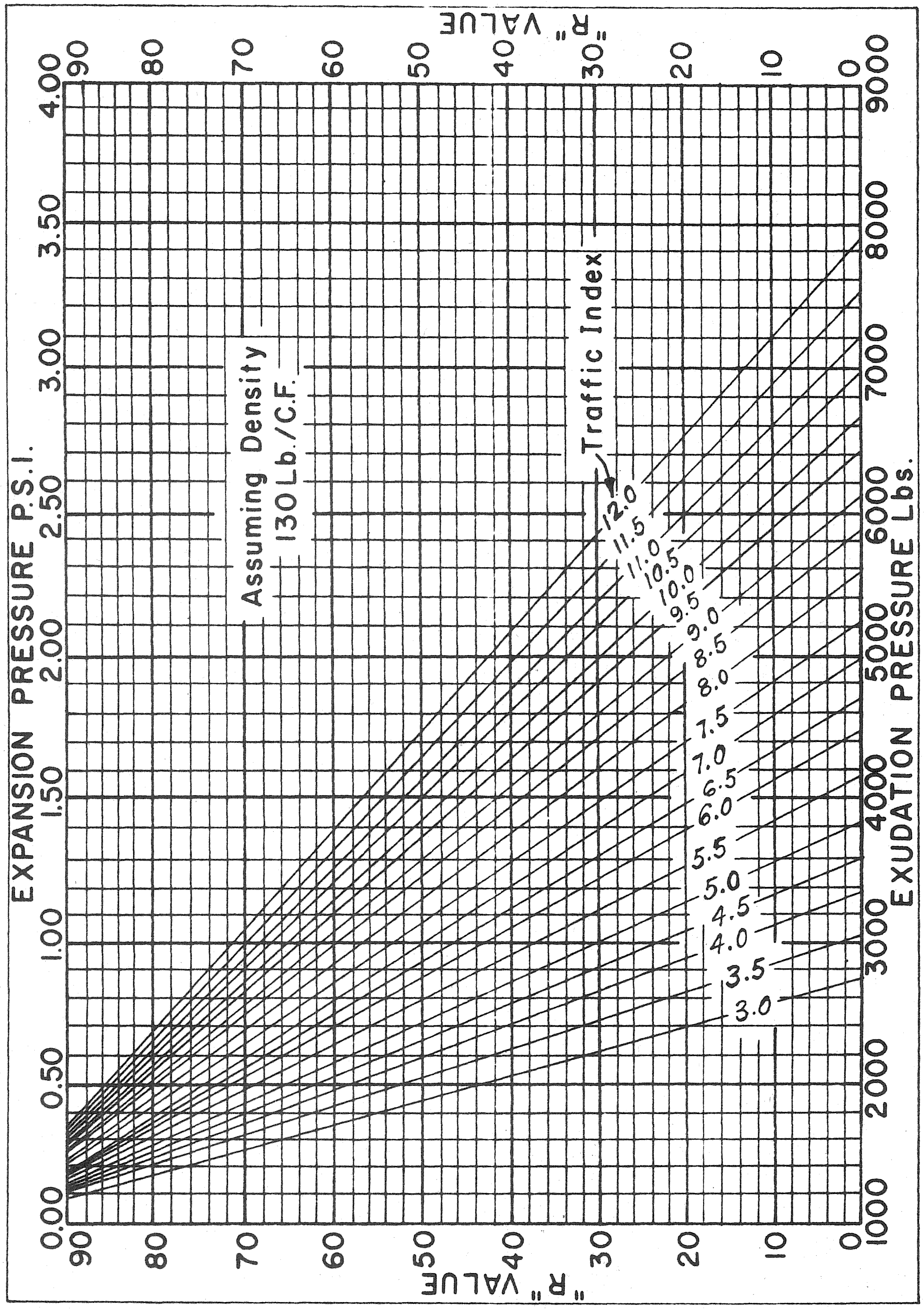


This report covers only material as represented by this sample and does not necessarily cover all soil from this layer or source.

Date Mailed 7/3/68

P. E.
Materials Engineer

EXPANSION PRESSURE BALANCE LINE



SELECT THE PROPER BALANCE LINE ACCORDING TO TRAFFIC INDEX, AND TRANSFER TO FORM DH-803.

GRADEPOINTS (16-232)

16-232.1 General

Gradepoints are defined as the roadbed transition from a cut to fill, generally though soils having been cultivated or having a higher organic content than underlying soils. Gradepoints are potential areas of weakness and must be examined critically before the pavement is designed. Factors which contribute to pavement failure at gradepoints are:

(1) Availability of Water: Melting snow, ponded runoff groundwater and capillary moisture all provide water to the gradepoint area. Adequate drainage, both surface and subsurface, is essential.

(2) Soil Type: Silty soils in the A-4 and A-5 category or under the Unified Soil Classification System under ML or MH category are the most susceptible to frost action when moisture is available. These soils also lose support very easily when saturated. Any soil which has been well cultivated or is capable of supporting abundant plant life should be considered susceptible to frost action.

(3) Frost: The factors of water and soil, taken together with frost action, cause greater loss of support to pavements than can be predicted by laboratory strength tests. Cycles of alternating freezing and thawing seem to cause the greatest damage. Moisture migrates from warm to cold areas, hence it is drawn up into the pavement structure.

16-232.2 Criteria for Treatment

Selecting those gradepoints which require treatment is largely a matter of judgment based upon consideration of the factors above. The following criteria shall be used in treating those selected:

(1) Remove topsoil to a depth below top of pavement equal to:

a. Pavement structure thickness, including all base courses, plus thickness of detrimental soil layer, or

Pavement structure thickness, plus one foot, whichever is least. Ordinarily the depth of the detrimental layer will vary from 0.5 to 1.0 feet. Exceptions may be highly organic silt soils which should be removed to a depth at least 1/2 the depth of frost penetration.

b. The length of treatment will be determined by the extent of the detrimental layer. Designation of limits on the plans should be in minimums of 50 foot increments rather than more refined limits.

(2) Remove all fine grained soil overlying sand, gravel or rock to a depth below top of pavement equal to the pavement structure thickness plus depth of soil not to exceed one foot.

(3) Backfill of rock excavation in cuts should only be with granular borrow or diced shot rock. Do not use any soil as it will entrap water and cause failure during spring breakup and a weakened section all year around.

(4) Replace excavated soil with clean granular borrow. Such material may be imported or it may come from selective use of rock or gravel excavation. In any case, it should be free-draining.

(5) Provide for adequate drainage, even if this requires deeper excavation. Water must not be trapped in the backfill material, but must be dissipated in the ditches or on the embankment slopes.

16-232.3 Typical Gradepoints and Treatment

Longitudinal and transverse sections. (See Figures 16-232.31 and 16-232.32). Gradepoint depths are prescribed in materials reports and letters. They are to be measured from finished grade.

SUBGRADE BLANKET (16-233)

16-233.1 General

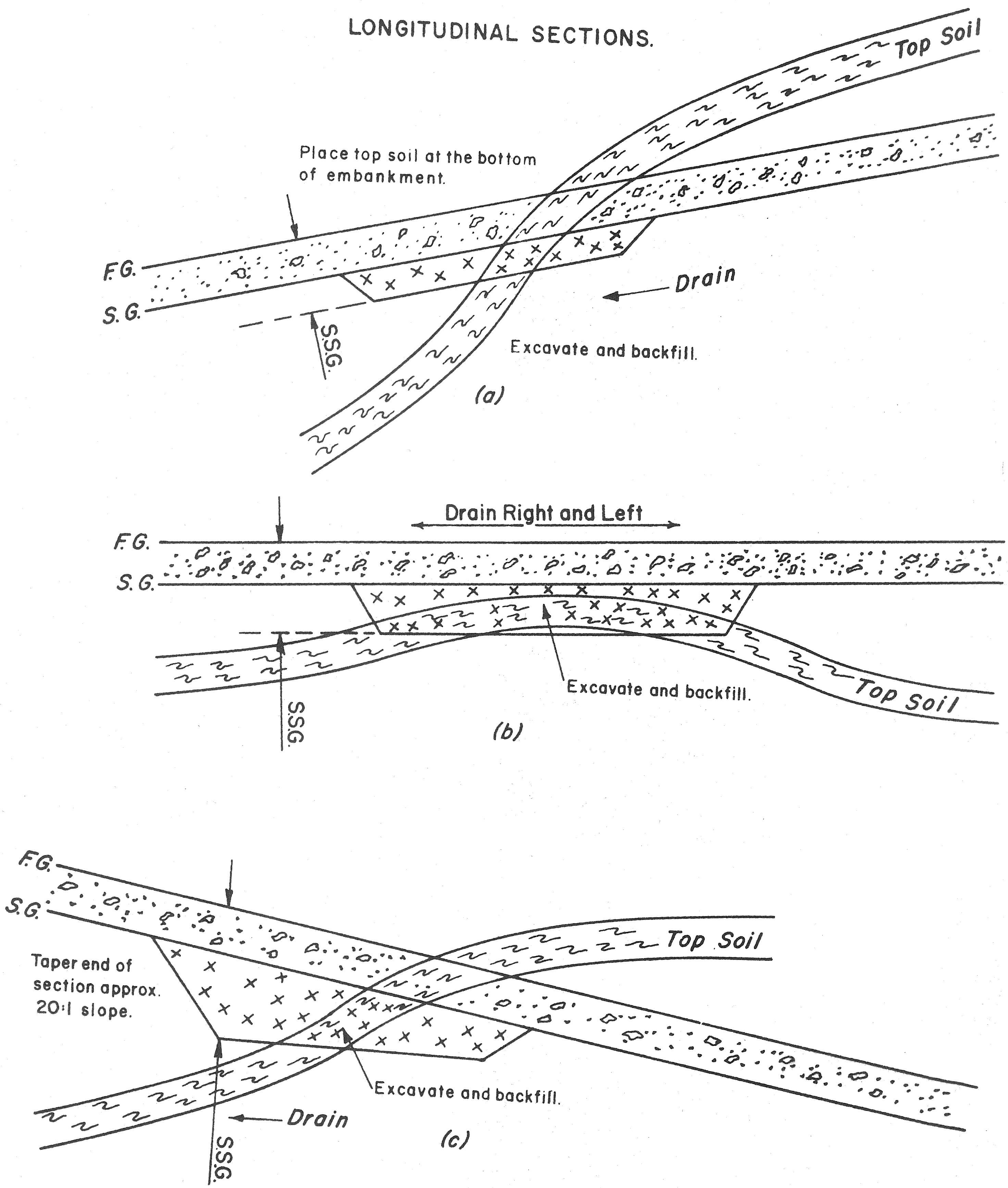
It is sometimes necessary to design for a blanket course of granular material to be placed over the subgrade soil. Soils which require a blanket are those fine-grained, plastic soils which, when water-saturated, tend to permeate and thereby contaminate otherwise free draining subbase or base materials under the action of traffic loads.

Laboratory criteria for requiring a blanket course on the soils evaluation summary are as follows:

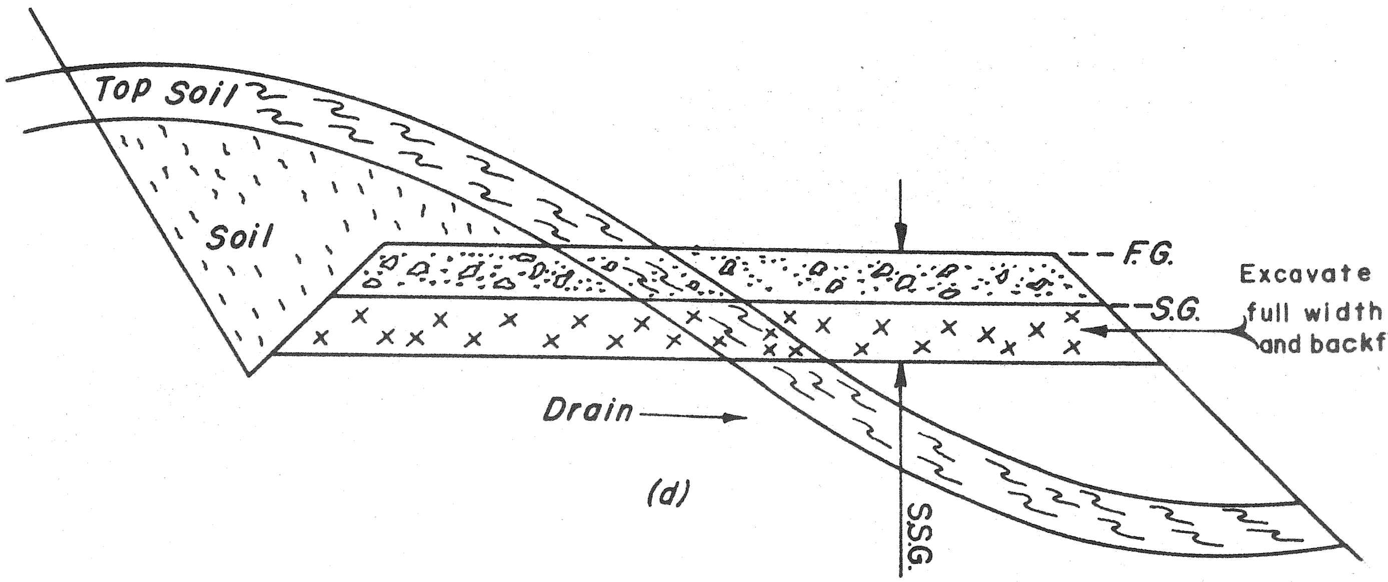
A blanket course is considered necessary if the soil tests show more than 65 percent passing the No. 200 sieve, and if the plasticity index and/or linear shrinkage values are equal to or greater than 5 percent. The requirement for blanket applies only if the soil appears at subgrade.

Granular material which is well graded and contains at least 15 percent minus No. 40, and 25 percent minus No. 10, is suitable blanket material. A blanket course is usually placed 0.25' in thickness.

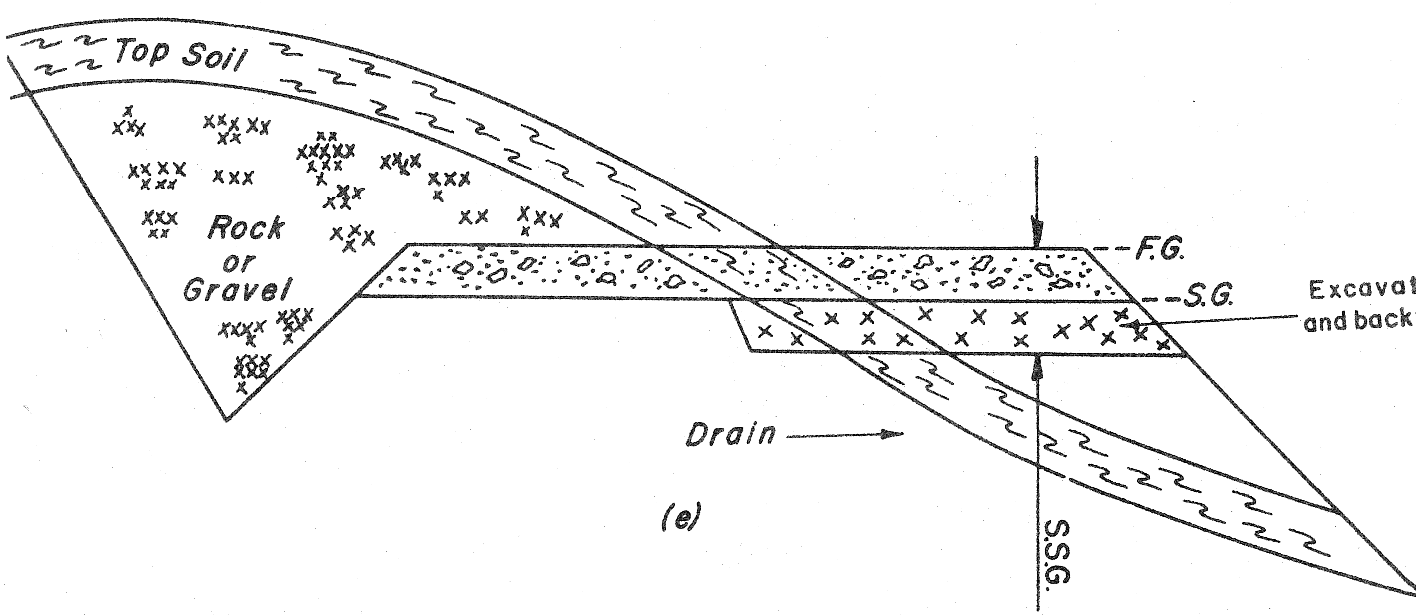
LONGITUDINAL SECTIONS.



TRANSVERSE SECTIONS.



(d)



(e)

DESIGN OF ASPHALT SURFACING OVERLAYS FOR ASPHALT PAVEMENTS (16-235)

16-235.1 Summary of Factors for Design

The major factors to be considered in designing an adequate overlay for existing asphalt pavement structures are:

1. Structural Quality of Existing Pavement

   The existing pavement structure may have been constructed of materials that have degraded or otherwise become deficient in quality. Stripping and cracking of the surfacing course can impair the structural quality. Samples should be taken of the subgrade for resistance values determinations, of the base materials for gradation, sand equivalent and resistance value tests; and of the surfacing course to determine the amount of stripping; asphalt content and apparent brittleness of the mixture.

2. Traffic

   Traffic volume increases since the time of construction, particularly of heavy trucks need to be carefully evaluated. Classification of traffic in accordance with Section 16-231.2 should be made. Should the overlay be designed to provide for less than a 20-year design period the traffic index shall be reduced appropriately.

3. Climatic Factors

   Section 16-231.5 provides for additional protection against winter and spring weather by providing additional thickness. These factors are to be provided when basing the design on resistance value and expansion pressures.

4. Deflections Under Wheel Loads

   It is possible that the structural design based upon resistance value, expansion pressure and, climate can be met and that higher than normal deflections are prevalent on the project due to resilient subgrade soils or base course materials. Measurement of deflections by means of the Benkelman beam (Idaho T-92) shall be made and plotted to determine the range of values for sections of the pavement and used to determine the overlay required to reduce the deflection to tolerable limits.

5. Economic Factors

   A satisfactory section can be designed using various combinations of materials. Selection of the best combination must consider elimination of porous sandwichs, layer thicknesses, continuity of sections available materials and the design period if less than 20-years.

16-235.2 Structural Quality of Existing Pavement Structure

1. The condition of the existing pavement structure and the cause of any deficiencies must be determined. A condition survey is desirable and should reflect section by section the actual condition of the pavement structure including:

    a. Cracked and patched areas including descriptive classifications of the cracking.

    b. Benkelman beam deflection measurements if available and limits of the comparable sections.

    c. Typical soil and base course test results showing Resistance values, percent passing No. 200 and sand equivalent of the base material and other significant quality characteristics.

    d. Thickness of surfacing and base.

    e. A roughness index determination is desired on pavement structurally sound but of sufficiently poor riding quality to warrant an overlay.

16-235.3 Overlay Design by Resistance Value Analysis

An analysis is made of the existing pavement structure, resistance values of subgrade and base course materials, expansion pressures, traffic analysis, climatic factors as required by Section 16-231. The following additional factors shall be given consideration:

1. Overlays planned for periods of less than 20 years shall have the traffic index selected for the reduced period proportioning the commercial traffic in accordance with the present and projected annual daily traffic.

Example:

$$\text{Commercial One Way ADT (Reduced) (Period)} = \frac{\text{Reduced Design Period}}{20} \times \frac{\text{Present Commerical One Way ADT} + \text{Proj. 20 yr. Commercial One Way ADT}}{2}$$

Traffic Index selected for the CADT (Reduced period using Figure 16-231.2).

2. Substitution Ratios of Existing Pavement Materials

    a. Base course materials are expected to meet quality requirements of present day specifications, ie, sand equivalent, Resistance value, percent passing No. 200.

    Should major deficiencies occur a substitution ratio of 3/4:1 shall be used. Minor deficiencies should be evaluated 1:1 unless project performance has indicated a lesser value to be necessary.

b. Existing asphalt pavement surfacing shall be evaluated not more than that set forth in Section 16-231.6. Should the material be deteriorated by cracking, raveling or stripping a reduction in substitution ratio to a value approaching 1:1 is advisable. Normally a full reduction is unwarranted. Pavements which are rough but having sufficient live asphalt even though flushing shall be given full value for the pavement type.

Pavements that are badly cracked should be reduced in value proportional to the amount of cracking, ie, alligator or chicken wire cracking reduced to the same value as aggregate base, block cracking only partial reduction in value, transverse temperature cracking should not be cause for any reduction in value.

c. Granular borrow or cinder aggregates shall be evaluated at 3/4:1 unless R-value determination would indicate a lesser value.

3. Climatic Factors

Increased structural requirements for severity of climate was not utilized in designs prior to 1965. After determining the required gravel equivalent increased thicknesses in accordance with Section 16-231.5 are to be applied.

16-235.4 Overlay Design By Deflection Analysis

The magnitude of pavement deflection is an indicator of the pavement structures ability to withstand traffic loading. Should the resistance value analyses fail to indicate any reasonable cause for the evident pavement distress an analysis of pavement deflections should be made. Pavement deflections shall be measured using Idaho T-92. The Determination of Load Deflection Characteristics of Flexible Pavements Employing the Benkelman Beam (WASHO Deflection Test Procedure).

16-235.41 Determination of Project Deflection Value for Selection of an Overlay Design

The condition survey shall be used to select sections of equal or nearly equal performance. Sufficient deflection measurements shall be made within each section to determine the normal distribution and variance of deflection values. The variance or range of values may indicate that sections of shorter or longer length may be selected. A plot of the deflection values is helpful in selecting sections. It is desired to keep the range of deflection values small within each section but it is also desirable to make each section as long as possible due to construction difficulties in adjusting thicknesses. Isolated points of weakness might be considered for separate treatment or reinforcement.

A value for the mean plus two standard deviations shall be determined and used for checking the adequacy of the pavement section. The procedure to be used is as follows:

1. Using not less than 10 random deflection measurements if taken at a local site (within 300 feet) or all measurements within a section of substantially equal performance and deflections, compute the mean (average) deflection. The standard deviation of the deflection values is as follows:

$$S.D. = \sqrt{\frac{n(\Sigma x^2) - (\Sigma x)^2}{n(n-1)}}$$

S.D. = Standard deviation
n = number of test values
x = individual test value

2. Determine the deflection value for the section to be used for overlay design purposes as follows:

Deflection = Mean deflection plus 2 standard deviations

16-235.42 Selection of Overlay Thickness (Asphalt Institute)

Figure 16-235.42 gives overlay thickness in feet for differing traffic indexes and deflections.

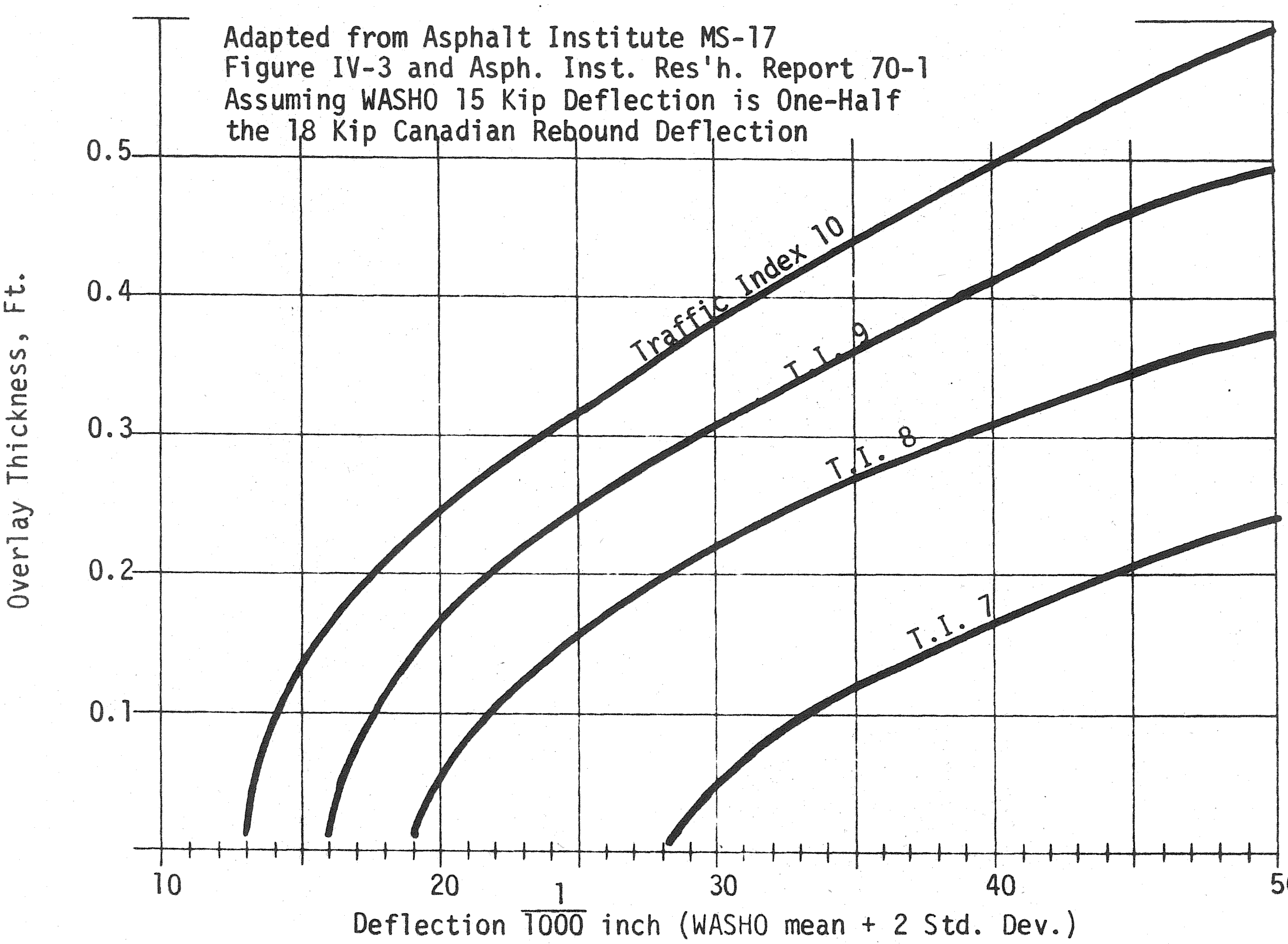


Figure 235.42

16-235.5 Example of Design

Example 1. Overlay thickness selection by Resistance Value Analysis

	1970 Data	One Way	1990 Projections	One Way
Total ADT (Heavy Classification)	3280	1640	7080	3540
Commercial ADT (% C=15.1)	495	248	1069	535

Commercial One Way ADT (Reduced Period =

$$\frac{\text{Reduced Design Period}}{20} \times \frac{\text{Present Comm. One-Way ADT} + \text{Proj. 20 yr. Comm. One-Way ADT}}{2}$$

$$\text{CADT (Reduced)} = \frac{8}{20} \times \frac{248 + 535}{2}$$

CADT (Reduced) = 156

Figure 16-231.2 T.I = 9.0

0.4' of aggregate base material meets today's quality requirements. The plant mix pavement is badly alligator cracked, therefore, use substitution ratio of 1:1.

Subgrade soils are represented by R-value of 40 with climatic factor = 1.0 (See Figure 16-231.3) Design thickness = 1.8'. The existing total flexible pavement thickness would have a 1.2' gravel equivalent. (0.4' plant mix pavement x 1.0=0.4', 0.4' plant mix base x 1.0 = 0.4', 0.4' aggregate base 3/4" x 1 = 0.4').

Overlay thickness by R-value = 1.8' - 1.2' = 0.6' gr. eq. 0.6' gr.eq/2 = 0.3' plant mix overlay (See Figure 16-231.6 for substitution ratios.)

Expansion pressure = 0.94 p.s.i. Figure 16-231.41 requires 1.0' ballast depth, therefore, existing design satisfies the expansion pressure requirement.

Example 2. Overlay Thickness Selection by Asphalt Institute Method.

Use T.I. = 9.0 as determined in Example 1. Use project deflection = .038" as determined below:

Project deflection (1/1000 inches)

X	X^2
24	576
26	676
26	676
28	784
30	900
30	900
30	900
32	1024
32	1024
34	1156
34	1156
34	1156
36	1296
$\Sigma X = 396$	$\Sigma X^2 = 12224$

$n=13$ $\bar{X} = \frac{396}{13} = 30.46$ or 31

$$\text{Std. Dev.} = \sqrt{\frac{n(\Sigma X^2) - (\Sigma X)^2}{n(n-1)}} = \sqrt{\frac{13(12224) - (396)^2}{13(13-1)}}$$

$$\text{Std. Dev.} = \sqrt{13.43} = 3.665 \text{ or } 3.7$$

$$\bar{X} + 2(\text{S.D.}) = 30.46 + 2(3.67) = 37.8 \text{ or } 38$$

See Figure 16-235.42 and with the above data a 0.4' plantmix overlay is obtained.

16-235.6 Selection of Overlay Thickness for Construction

It is not to be expected that the overlay thicknesses determined by the two methods herein described will check exactly. Resilience or elasticity of subgrade and base materials may cause deflections to be greater than normal. Also deflection measurements may not be measured during the critical period and, therefore, are less than they might have been. It is, therefore considered that the values obtained express a range of thicknesses to be evaluated for cause of defeciencies in the existing pavement structure. The values also indicate the effectiveness of the overlay in correcting the defeciencies. The final selection of thickness must balance these considerations, available funds, traffic volume and service life expected.

IDAHO FLEXIBLE PAVEMENT DESIGN

Material Equivalencies

The WASHO and AASHO road tests established the validity of the strength equivalencies of various materials used in a pavement structure. Many mathematical expressions were developed by the HRB staff conducting the AASHTO test, the Asphalt Institute, Portland Cement Association and others.

The AASHTO Road Test evaluation gave values of equivalencies (inches of stone base per inch of treated base) to vary from about 1.5 to 3.3 for the various materials used. The California Department of Highways using coast asphalts and local aggregates when compared to AASHTO road test materials got values from 1.5 to 2.5 The Asphalt Institute analyzed the AASHTO road test data independently by first transforming the multilayered AASHTO Design into a single parameter using substitution ratios or equivalencies of subbase gravel, stone, CTB of Bitumnous treated base for esch inch of asphalt concrete. Regression analysis were made resulting in an equivalency of 2 for untreated granular base.

Since the WASHO and AASHO road tests were of short duration, only two years, it was decided to use the more conservative equivalencies developed by California and the Asphalt Institute in Idaho Design procedures.

Since low volume roads on county and local highway district secondaries are difficult to finance, larger ratios are permitted. A Traffic Index of 5.4 and 6.9 is used to differentiate these roads. The 6.9 TI gives from 30 to 100 commercial vehicles daily or an ADT of from 600 to nearly 2000. Table I in the Highway Department manual gives these equivalencies.

IDAHO FLEXIBLE PAVEMENT DESIGN:

CLASS PROBLEM

GIVEN:

	1975	1995
ADT	2500	6000
Commercial ADT	375	800
% 2 Axle	40%	
% 5 Axle	30%	
Classification		
Subgrade R value	23	
Expansion Press, PSI	1.23 psi	
Wt/CF Base & Surf.	141	
Climate Zone	3	
Design Std.	2-Lane Primary	

Determine Pavement Design Classification and Structural Sectons using:

(1) 0.25 Asph Conc.
0.5 CTB
Gravel Base

(2) 0.4 Asph Conc.
Gravel Base

Which is most economical Section/SY of travel lane if estimated cost at construction are

Asphalt Concrete	10^{00}/ton
CTB	5^{00}/ton
Gravel Base	2^{50}/ton

APPENDIX A

PROPOSED ADDITION TO THE MATERIALS AND RESEARCH MANUAL DESIGN FOR CONCRETE PAVEMENT (16-232)

16-232.1 INTRODUCTION

The usual procedure for Portland Cement Concrete Pavement design involves a detailed analysis of concrete stresses due to load. It incorporates considerations of the number and distribution of axle loads in the design life, subgrade support represented in terms of K factor, and flexural strength of the concrete.

Using these data, and design charts which show stresses for single and tandem axle loads, a "% fatigue used" is calculated. This enables the Engineer to select a pavement thickness.

The Materials Section has incorporated the elements of this procedure in a pavement thickness design nomograph. The nomograph correlates the above data with parameters common to our flexible pavement design: Traffic Classifications, Heavy, Average, Light; R-value and Idaho axle load data from W-4 Table.

This allows the designer to determine a thickness of flexible or rigid pavement from the same basic data with a minimum of effort and conversion.

16-232.2 STRENGTH OF CONCRETE

The completeness of cement hydration which produces strength in the cement paste determines the strength of any given concrete mixture. If moisture is available, hydration continues for an indefinite length of time. The quality of aggregate particles also contributes to the strength or weakness of the concrete, but since the aggregate particles are stronger per unit area than the hardened paste the strength of the concrete varies with the quality of the hardened paste.

Laboratory tests on the concrete aggregate report the flexural strength at 14 and 28 days.

Idaho specifications require a minimum flexural strength for concrete pavements by laboratory produced and cured specimens to be not less than 550 psi in 14 days. The design procedure described here includes a design figure for Modulus of Rupture determined by the third point loading in 28 days. Obtain Modulus of Rupture (28 day MR) from data from tests on aggregate to be used on the particular project.

16-232.3 SUBGRADE SUPPORT

The second major element in concrete thickness design is subgrade support. The subgrade support is expressed by the R-value which is a test value which measures the ability of a soil to resist lateral flow due to vertically applied loads. The Rv can be obtained from our soil lab reports. By the use of a graph which was developed by the California Highway Department, R-value can be converted to K factor. See Figure 16-232.31.

This K factor then becomes Ks (the K at the top of the basement soil layer which would normally be directly below the granular borrow, base material, or cement treated base).

The Ks is converted to Kf by Figure 16-232.32. Kf is the K factor or subgrade support value at the top of aggregate base or granular borrow. Start at the bottom of Figure 16-232.32 with the thickness of subbase, then with Ks on the basement soil, the Kf on the top at the subbase is determined at the left side of the figure. Likewise using Figure 16-232.33 and with the Kf on the subbase and a given thickness of cement treated base, the Kc on the top of the CTB can be determined.

Figure 16-232.34 is to be used when a Plant Mix Base is used in place of a Cement Treated Base. This would apply when a concrete pavement overlay over an existing Plant Mix pavement is to be constructed or when the project exhibits the need for stage construction with a plant mix wearing course because of possible embankment settlement. The concrete pavement would be stage constructed at a later date after critical embankment settlement.

Kc is referred to as a combined K. The term "combined" refers to a K factor which is increased by the additional structural support of untreated or treated base.

If the Rv is known and the Ks factor has been determined from Figure 16-232.31, there is no subbase or C.T.B., then Ks would become Kc and used directly in the thickness nomograph, Figure 16-232.35. Normally we would not eliminate the C.T.B. but this has been cited to better explain the use of these figures.

16-232.4 <u>LOAD SAFETY FACTORS</u>

In using the conventional design procedure axle weights are increased by a certain amount according to the type of facility and character of traffic. This factor is known as a Load Safety Factor (LSF) and has been included in this design procedure but is not shown as a separate parameter in the thickness design nomograph.

16-232.5 <u>LOAD STRESSES</u>

With virtually all traffic moving along the slab interior, the critical stress location is at the transverse joint edge. In Case I single and tandem-axle loads are at the transverse joint edge. Maximum flexural stresses occur at the bottom of the slab and are parallel to the joint edge. Case I design charts for single and tandem-axle loads were used in this design method. Case I, II and III are explained in detail on Page 6 in the Portland Cement Association Booklet entitled "Thickness Design for Concrete Pavements".

16-232.6 TRAFFIC EVALUATION

See the discussion under Subsection 16-231.2 for the derivation of Traffic Index (T.I.).

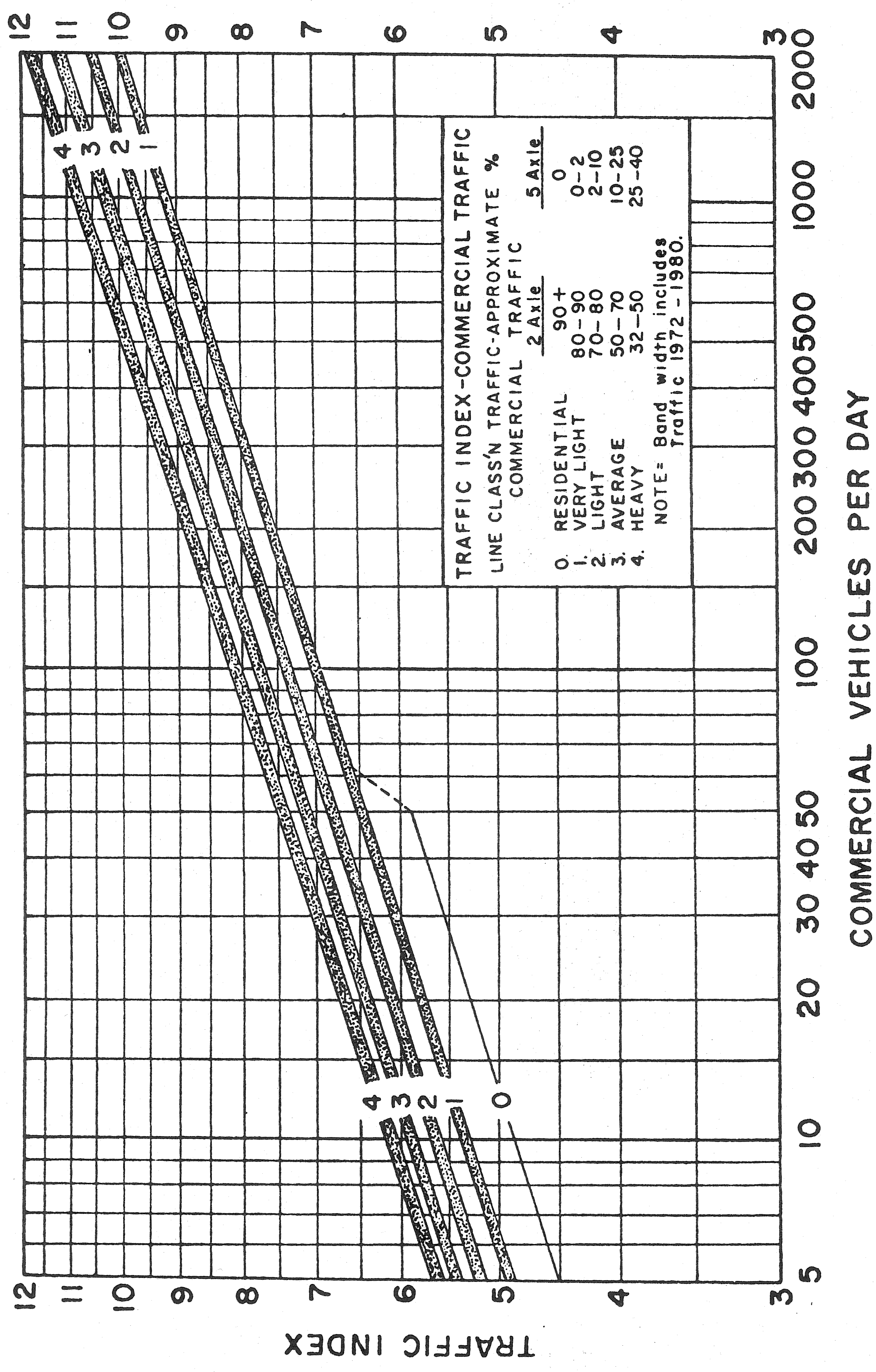
16-232.7 EXAMPLE OF DESIGN

To illustrate a concrete pavement thickness design assume the following data:

Average CADT (one way)	210
Traffic Index (T.I.)	9.0
Traffic Classification	Heavy
Design Life	20 Years
Depth of CTB	0.33'
Depth of Granular Borrow	0.5'
Modulus of Rupture (MR)	600 PSI
R Value (Rv) at top of Soil Layer #1	25

Determine the Ks at the top of the soil from Figure No. 16-232.31, Ks = 85.

With Ks and Figure No. 16-232.32 determine the Kf at the top of the granular borrow, Kf = 110.

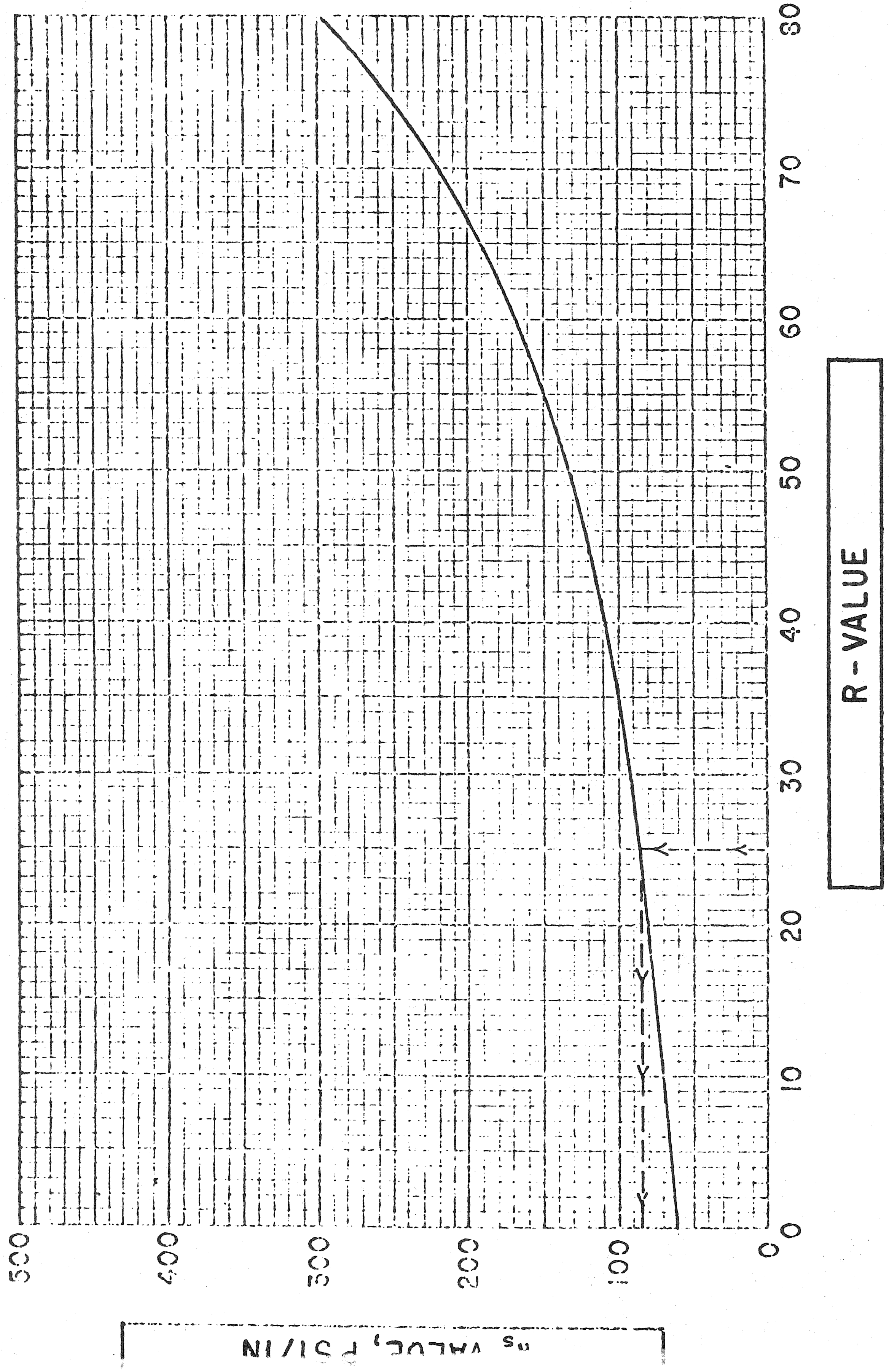


TRAFFIC INDEX FROM COMMERCIAL VEHICLE COUNT & CLASSIFICATION

With Kf and Figure No. 16-232.33 determine the Kc (combined K) at the top of the CTB, Kc = 220.

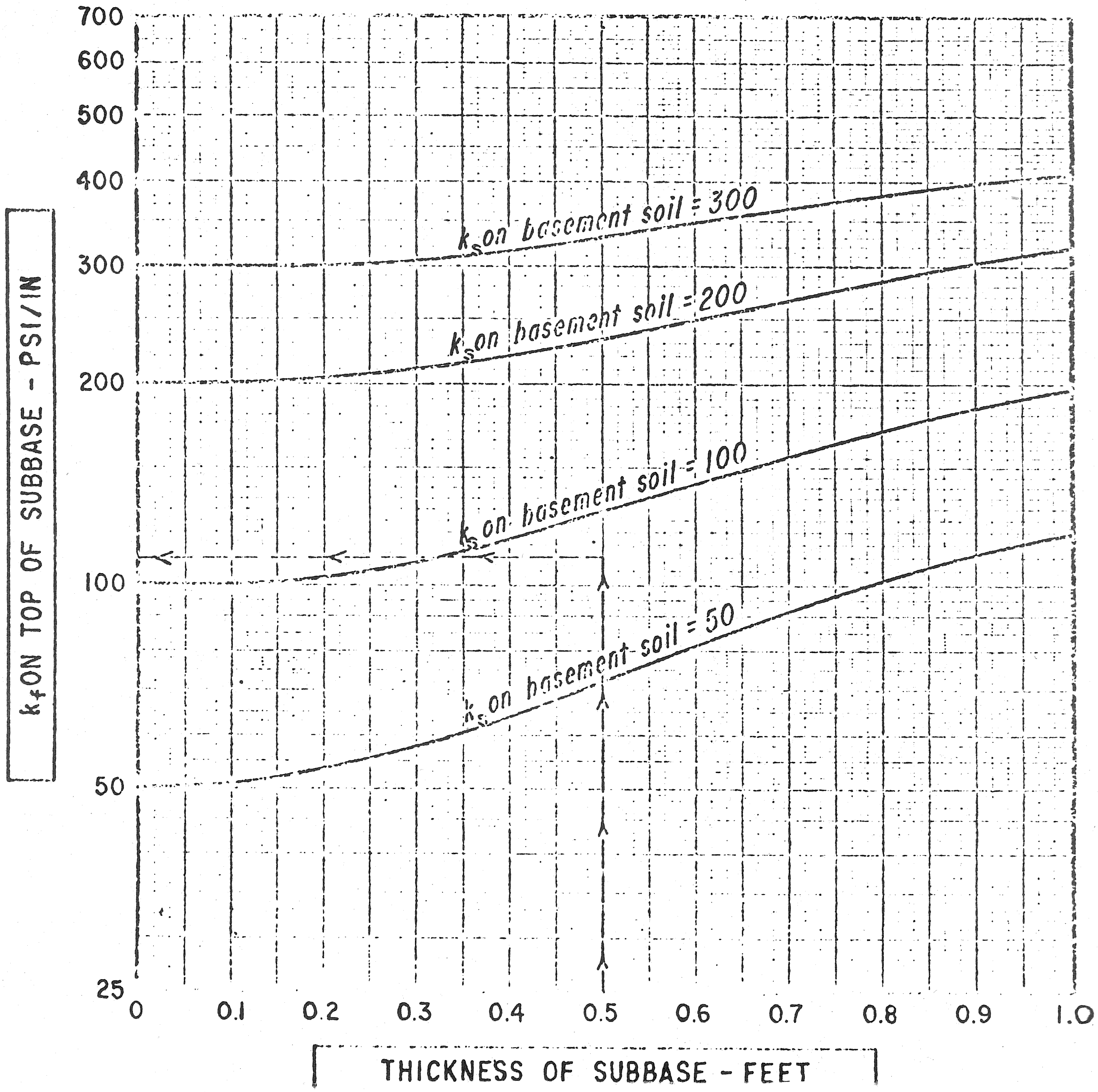
Using Figure No. 16-232.35 and the above data, a thickness between $7\frac{1}{4}$" and $7\frac{1}{2}$" is required, taking the thickness to the closest $\frac{1}{2}$" use a design thickness of $7\frac{1}{2}$". If the nomograph had indicated less than $7\frac{1}{4}$" then 7" should be used.

Fig. 16-232.31

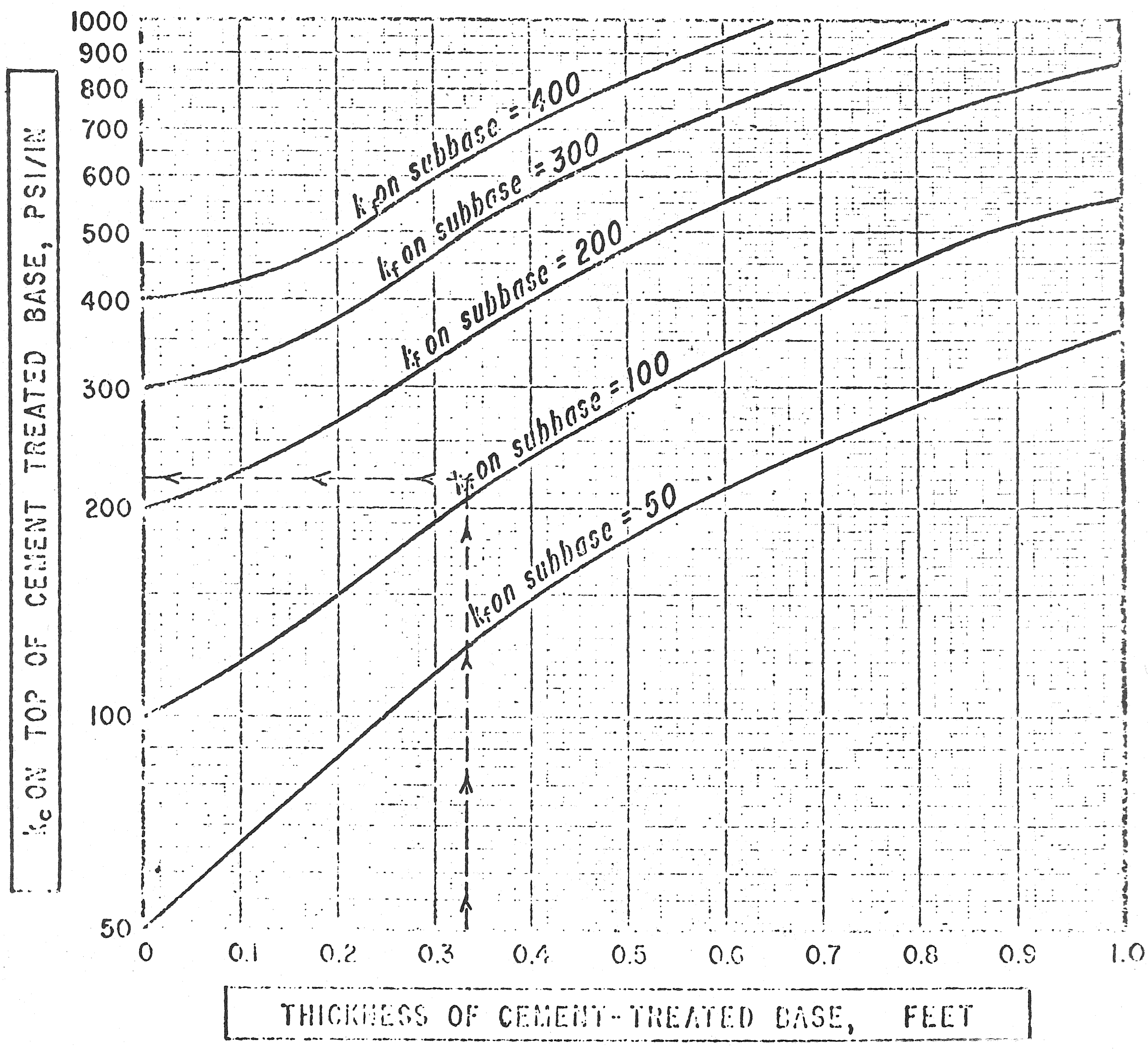


K-VALUE VS. R-VALUE

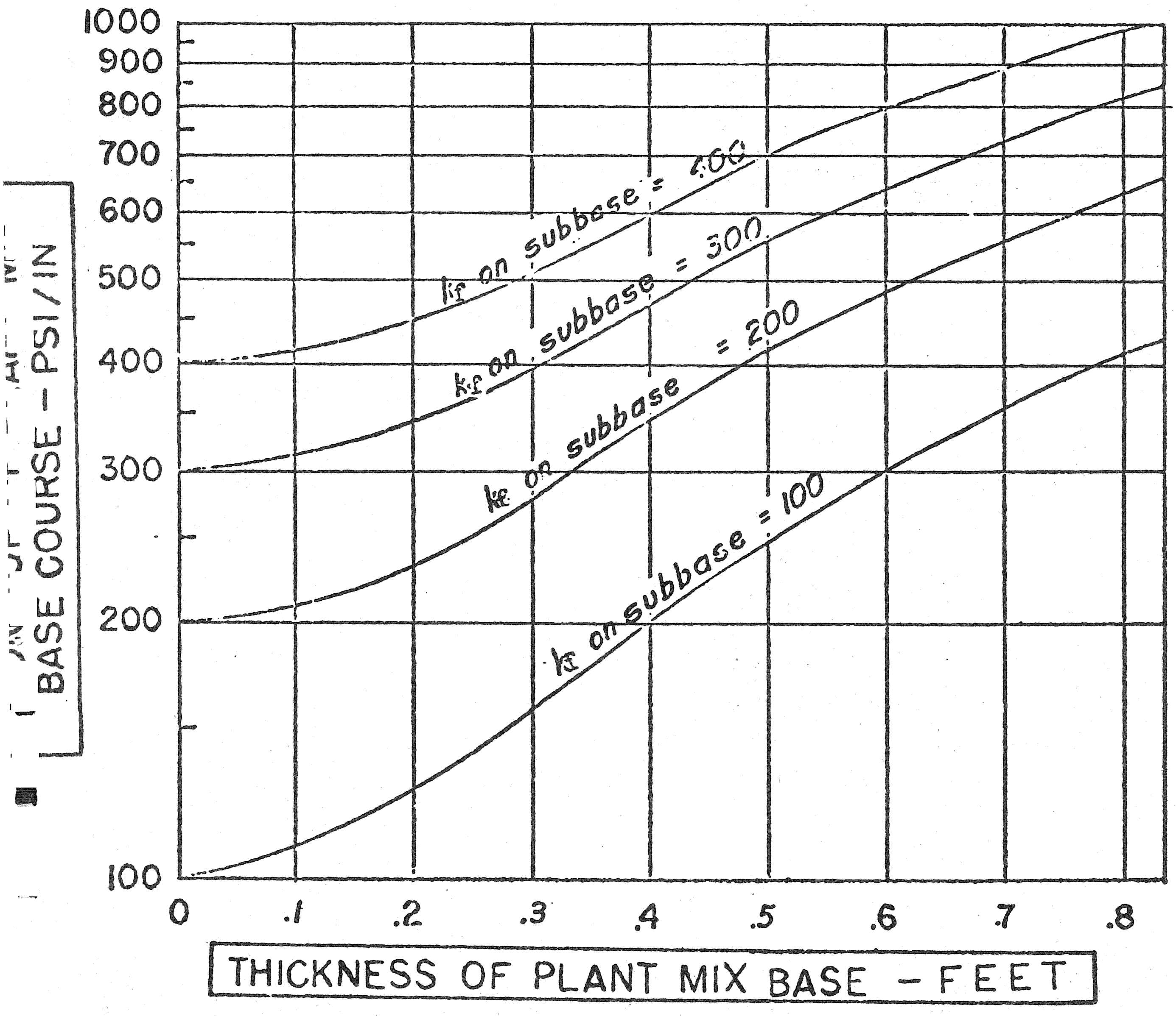
EFFECT OF VARIOUS THICKNESSES OF PLANT MIX BASE ON k VALUES



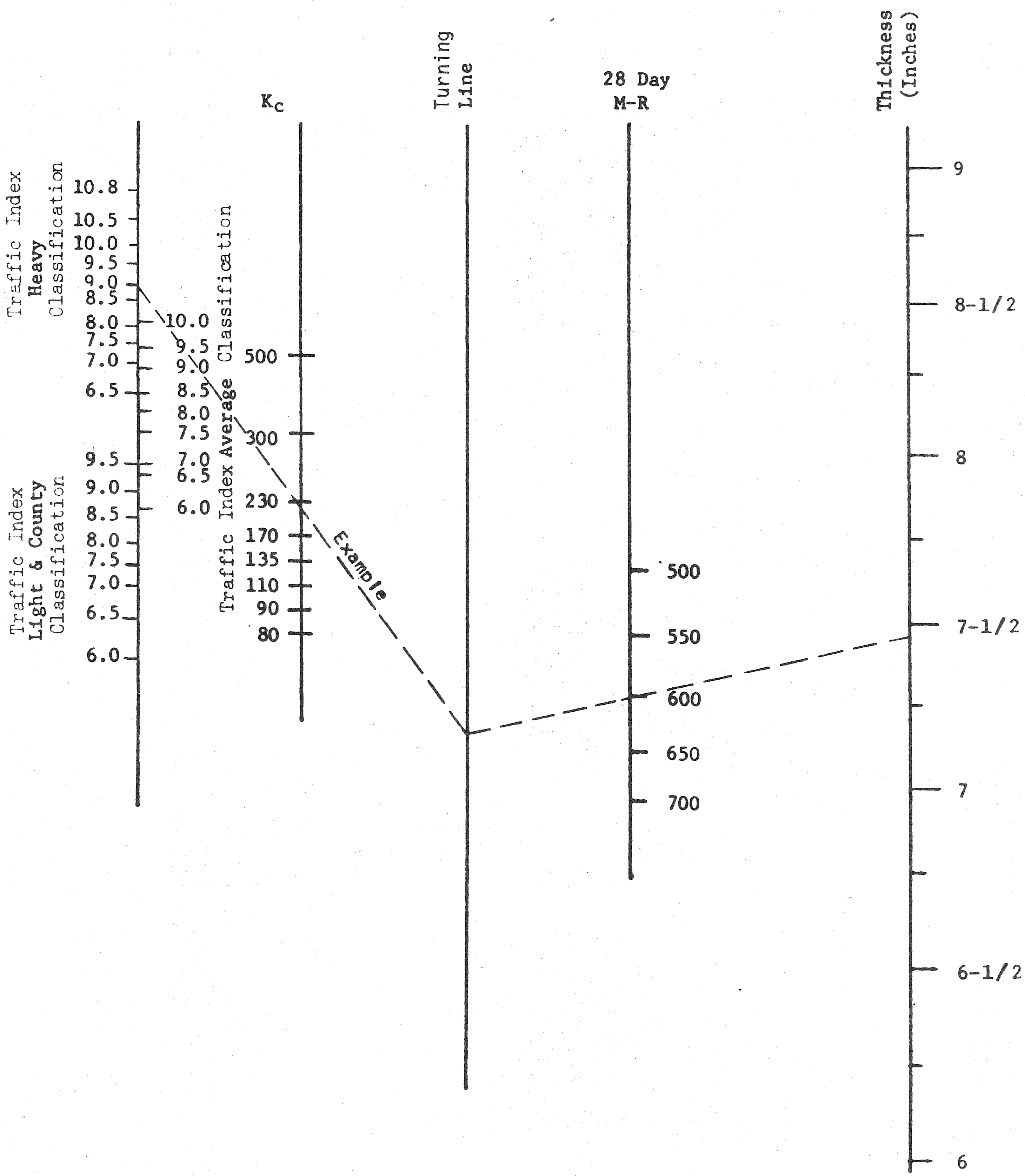
EFFECT OF VARIOUS THICKNESSES OF PLANT MIX BASE ON k VALUES



EFFECT OF VARIOUS THICKNESSES OF PLANT MIX BASE ON k VALUES



THICKNESS DESIGN CHART FOR CONCRETE PAVEMENT



Note: Xerox copies are distorted and will not provide an accurate thickness

PORTLAND CEMENT CONCRETE THICKNESS DESIGN

IDAHO METHOD

CLASS PROBLEM

Find the minimum thickness of p.c.c. pavement for the following conditions:

ADT = 5000; commercial = 10% or 500 CADT. Traffic Classification: "Heavy"

Subgrade R-value = 20

Subbase (base) = cement treated base = .33 ft.

Concrete: M-R = 600 psi @ 28 days.

REINFORCEMENT OF RIGID PAVEMENTS

Many western states including California, Washington and Idaho do not reinforce concrete pavements and use a treated base either CTB or ATB in lieu of dowels. Doweled joints are anything but trouble free.

Oregon has recently been constructing continuously reinforced concrete pavement believing that they will gain longer maintenance free life and overall durability of the pavement. Personal ovservation of these pavements shows that they are cracking although not faulting at the cracks. Any salting of these pavements may cause corrosion and problems in texture. Generally a CRC Pavement will crack transversely about every 3 to 5 feet. Cracking on the Idaho job is in this range.

PAVING MATERIALS AND TESTS

Aggregates -

Generally the same quality aggregates are used in surfacing and base courses. The aggregate is required to meet specific gradations and is the hard, durable, and free of organic materials and soft particles.

	Bituminous Mixtures	Untreated Base Matls	Sub-Base Matls
Gradation	X	X	X
Wear on Coarse Matl.	X	X	
Sand Equivalent	X	X	X
Atterberg Limits	X	X	X
Soundness Tests	X	X	
Specific Gravity Bulk	X		
Rice Sp Gr of Paving Mix	X		
Absorption Water	X	X	
Unit Wt/CF Compacted	X	X	X
Asphalt Absorption	X		
Mix Design Asphalt	X		
Stripping Characteristics	X		
Degradation of Aggregate	X	X	X

Gradation -

Gradation charts for 1/2" Max, 3/4" Max and 1" Max type IV mixes of the Asphalt Institute are attached. Gradation of the aggregate should provide adequate density,(voids in the mineral aggregate from 15-18 percent) and limit the amount passing the No. 200 sum to no more than 8-10 percent. The Sand Equivalent should be above 35 for normal plant mixed and base material and preferably above 50.

Wear -

Normally a Los Angeles Wear test is made on coarse aggregate. It is a measure of hardness and should not exceed 40 except when performance in service can prove satisfactoriness otherwise other wear tests may be run such as wet or dry Deval wear.

Sand Equivalent -

This test measures volumetrically the amount of clay or colloid material in a fine aggregate. Clean, washed concrete sand is 75+. Any clay will generally drop sand equivalent values to less than 30. Any montmorillonite clay, even 1%, can drop values to less than 30. Since materials which cause low SE values cause disintegration of bitumimous mixtures in presence of water the SE is specified high enough to prevent problems.

Atterberg Limits -

The Atterberg Limits, liquid limit and plasticity index are specified 25 and 6 maximum respectively. However a SE of 35+ generally (99.99%) will give values of LL less than 25 and non-plastic material so it is not often specified.

Specific Gravities -

Both bulk and Rice Specific gravities are desired for bitimimous surfacing in order that asphalt absorption can be calculated.

Absorption in water is an indicator of the porosity of aggregate. If too porous, it may absorb too much asphalt or be unsound.

Soundness -

Many specifications specify sulpahte soundness tests which is a substitute for freeze and thaw tests. It does not always predict degradation especially in basalt or igneous rocks.

Stripping Characteristics -

Tests made on compacted mixtures by soaking in water. Unless water permeates the mixture, the test is of little value. Idaho has adopted vacuum saturation to assure water has a chance to react. The ratio of strengths soaked to unsoaked is a measure of the effect of water. Tests are generally made at 140^{o}F.

Degradation Tests -

Idaho, Oregon, Washington, California, and Alaska have tests to measure degradation. The degradation of concern is increased. Amounts passing the No. 200 sieve in presence of water. The wear test and soundness tests do not indicate this property. Idaho rotates a total sample of aggregate 10000 times in a glass jar under a small quantity of water. The increase in No. 200 and decrease in SE is measured.

Concrete Aggregates -

Concrete aggregates need to be hard, durable, and capable of producing concrete of the required strength. Some aggregates are structurally weak and even though sound will not preduce consistantly the highest strengths needed for pre-stress concrete.

Tests made are:

Gradation
Specific Gravity (bulk)
Absorption Water
Organic Color
Mortar Strength
Concrete Mixture Design

Only difference in tests are a Sodium Hydroxide test for organic impurities and mortar strength against Ottowa sand mortar. Actually Ottowa sand mortar is not a high strength mortar and therefore a 100% ratio is considered minimal. If high strength concrete is desired actual concrete mixes should be designed and tested to assure that quality concrete is possible from the conbinations considered.

Asphalt Cement -

AASHO provides two specifications for asphalt cement. M-20 for penetration graded material and M-226 for viscosity graded material. Penetration grading is the older specification. The viscosity graded specification provides three tables. Table I the least restrictive and unless one of the others is specified is the one to be furnished. Table 2 raises the limits of minimum penetraton allowable and hence is a less temperature susceptible asphalt. Table 3 calls for test limits to be met on the residue from thin film test

The selection of the asphalt grade for any particular project is becoming of greater concern to more and more engineers. The hard grades while resistant to displacement and shoving are frequently subject to low temperature cracking. Research has shown softer grades are not as subject to cracking. Canada has been a leader in this area.

The net result is that the softest asphalt cement should be used consistent with ability to resist displacement in the hottest summer months. Traffic plays an important role in the summer problem. Generally for Idaho it appears that the following grades can be used successfully.

	Winter Temp ^{o}F	
Traffic Count	0^{o}	-20^{o} or colder
Less 1000	AC 2.5,5 or 200/300	all
1000-4000	AC 5 or 200/300	AC 5 200/300
4000-plus	AC 10 or 120/150	AC 5 or 200/300

Standard Specification for

Viscosity Graded Asphalt Cement

AASHTO Designation: M 226-73

1. SCOPE.

1.1 This specification covers asphalt cements graded by viscosity at 60 C (140 F) for use in pavement construction. Three sets of limits are offered in this specification. The purchaser shall specify the applicable table of limits. In the event the purchaser does not specify limits. Table 1 shall apply. For asphalt cements graded by penetration at 25 C (77 F), see AASHTO M 20 for Asphalt Cement.

2. MANUFACTURE

2.1 The asphalt cement shall be prepared from crude petroleum by suitable methods.

3. REQUIREMENTS

3.1 The asphalt cement shall be homogeneous, free from water, and shall not foam when heated to 175 C (347 F).

3.2 The asphalt cements shall conform to the requirements given in Tables 1, 2 or 3, as specified by the purchaser.

4. METHODS OF SAMPLING AND TESTING

4.1 Sampling and Testing of asphalt cements shall be in accordance with the following standard methods of the American Association of State Highway and Transportation Officials:

Sampling	T 40
Viscosity at 60 C (140 F)	T 202
Viscosity at 135 C (275 F)	T 201
Penetration	T 49
Flash Point (COC)	T 48
Flash Point (P.M.C.T.)	T 73
Solubility in trichloroethylene	T 44
Thin-film oven test	T 179
Ductility	T 51
Spot test	T 102
Rolling Thin film oven test	T 240
Water	T 55

TABLE 1
Requirements for Asphalt Cement Graded by Viscosity at 60 C (140 F)
(Grading based on original asphalt)

TEST	VISCOSITY GRADE AC-2.5	AC-5	AC-10	AC-20	AC-40
Viscosity, 60 C (140 F), poises	250 ± 50	500 ± 100	1000 ± 200	2000 ± 400	4000 ± 800
Viscosity, 135 C (275 F), Cs-minimum	80	110	150	210	300
Penetration, 25 C (77 F), 100 g., 5 sec.-minimum	200	120	70	40	20
Flash Point, COC, C (F)-minimum	163(325)	177(350)	219(425)	232(450)	232(450)
Solubility in trichloroethylene, percent-minimum	99.0	99.0	99.0	99.0	99.0
Tests on residue from Thin-Film Oven Test:					
Viscosity, 60 C (140 F), poisees-maximum	1000	2000	4000	8000	16000
Ductility, 25 C (77 F), 5 cm per minute cm-minimum	100 [1]	100	50	20	10
Spot test (when and as specified)[2] with:					
Standard naphtha solvent	Negative for all grades				
Naphtha-Xylene-solvent, % Xylene	Negative for all grades				
Heptane-Xylene-solvent, % Xylene	Negative for all grades				

[1] If ductility is less than 100, material will be accepted if ductility at 15.6 C (60 F) is 100 minimum.

[2] The use of the spot test is optional. When it is specified, the Engineer shall indicate whether the standard naphtha solvent, the naphtha-xylene solvent, or the heptane-xylene solvent will be used in determining compliance with the requirement, and also, in the case of xylene solvents, the percentage of xylene to be used.

TABLE 2
Requirements for Asphalt Cement Graded by Viscosity at 60 C (140 F)
(Grading based on original asphalt)

TEST	VISCOSITY GRADE AC-2.5	AC-5	AC-10	AC-20	AC-40
Viscosity, 60 C (140 F), poises	250 ± 50	500 ± 100	1000 ± 200	2000 ± 400	4000 ± 800
Viscosity, 135 C (275 F), Cs-minimum	125	200	250	300	400
Penetration, 25 C (77 F), 100 g, 5 sec.-minimum	220	140	80	60	40
Flash Point, COC, C (F)-minimum	163(325)	177(350)	219(425)	232(450)	232(450)
Solubility in trichloroethylene, percent-minimum	99.0	99.0	99.0	99.0	99.0
Tests on residue from Thin-Film Oven Test:					
Loss on heating, percent-maximum		1.0	0.5	0.5	0.5
Viscosity, 60 C (140 F), poises-maximum	1000	2000	4000	8000	16000
Ductility 25 C (77 F), 5 cm per minute, cm-minimum	100 [1]	100	75	50	25
Spot test (when and as specified)[2] with:					
Standard naphtha solvent	Negative for all grades				
Naphtha-Xylene-solvent, % Xylene	Negative for all grades				
Heptane-Xylene-solvent, % Xylene	Negative for all grades				

[1] If ductility is less than 100, material will be accepted if ductility at 15.6C (60 F) is 100 minimum.
[2] The use of the spot test is optional. When it is specified, the Engineer shall indicate whether the standard naphtha solvent, the naphtha-xylene solvent, or the heptane-xylene solvent will be used in determining compliance with the requirement, and also, in the case of xylene solvent, the percentage of xylene to be used.

TABLE 3
Requirements for Asphalt Cement Graded by Viscosity at 60 C (140 F)
(Grading based on residue from Rolling Thin Film Oven Test)

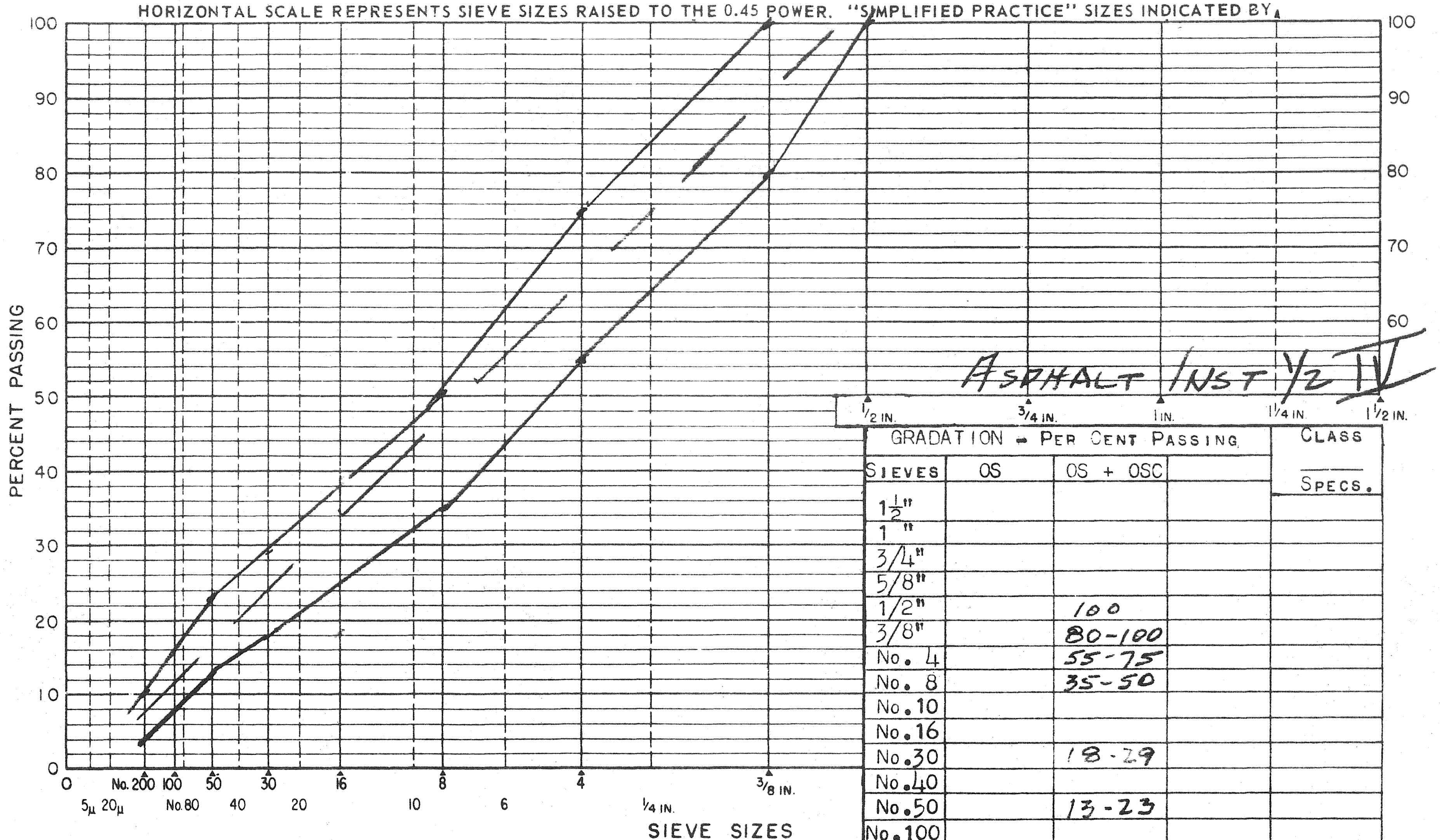
TESTS ON RESIDUE FROM AASHTO TEST METHOD T 240[1]	VISCOSITY GRADE AR-10	AR-20	AR-40	AR-80	AR-160
Viscosity, 60 C (140 F), poise	1000 ± 250	2000 ± 500	4000 ± 1000	8000 ± 2000	16000 ± 4000
Viscosity, 135 C (275 F), Cs-minimum	140	200	275	400	550
Penetration, 25 C (77 F), 100 g, 5 sec.-minimum	65	40	25	20	20
Percent of original Pen., 25 C (77 F)-minimum	—	40	45	50	52
Ductility, 25 C (77 F), 5 cm per min., cm-minimum	100 [2]	100 [2]	75	75	75
TESTS ON ORIGINAL ASPHALT					
Flash Point, P.M.C.T., C (F)-minimum	205(400)	219(425)	227(440)	232(450)	238(460)
Solubility in Trichloroethylene, percent-minimum	99.0	99.0	99.0	99.0	99.0

[1] AASHTO T 179 (Thin-Film Oven Test) may be used, but AASHTO T 240 shall be the referee method.
[2] If ductility is less than 100, material will be accepted if ductility at 15.6 C (60 F) is 100 minimum.

-958 10- [redacted]

GRADATION CHART

HORIZONTAL SCALE REPRESENTS SIEVE SIZES RAISED TO THE 0.45 POWER. "SIMPLIFIED PRACTICE" SIZES INDICATED BY ▲



GRADATION - PER CENT PASSING				CLASS / SPECS.
SIEVES	OS	OS + OSC		
1 1/2"				
1"				
3/4"				
5/8"				
1/2"		100		
3/8"		80-100		
No. 4		55-75		
No. 8		35-50		
No. 10				
No. 16				
No. 30		18-29		
No. 40				
No. 50		13-23		
No. 100				
No. 200		4-10		

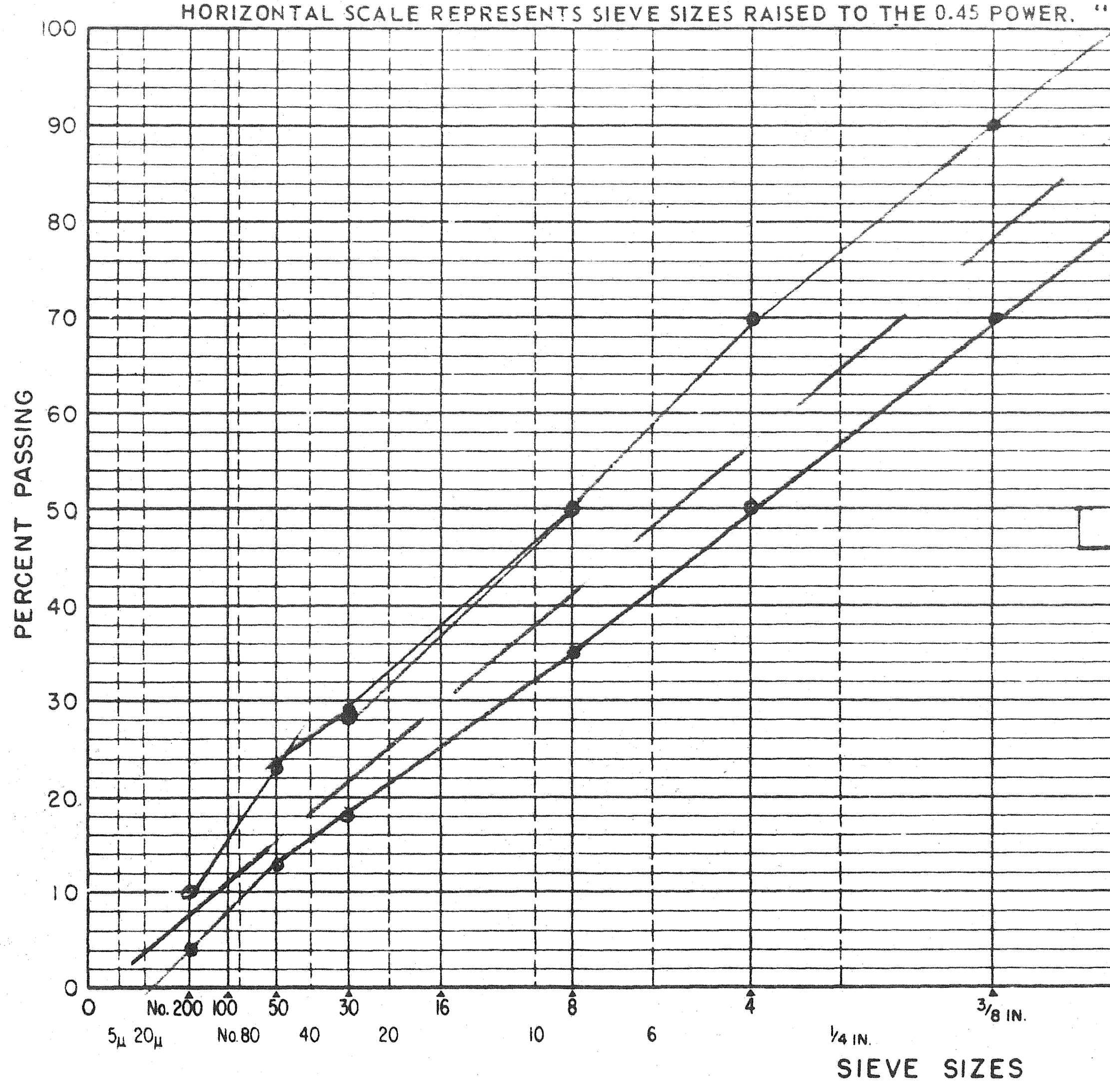
PROJECT ______________ LAB. NO. ______________

IDENT. NO. ______________ PIT. NO. ______________

-95 10-

GRADATION CHART

HORIZONTAL SCALE REPRESENTS SIEVE SIZES RAISED TO THE 0.45 POWER. "SIMPLIFIED PRACTICE" SIZES INDICATED BY ▲



ASPHALT INST - 3/4 IV

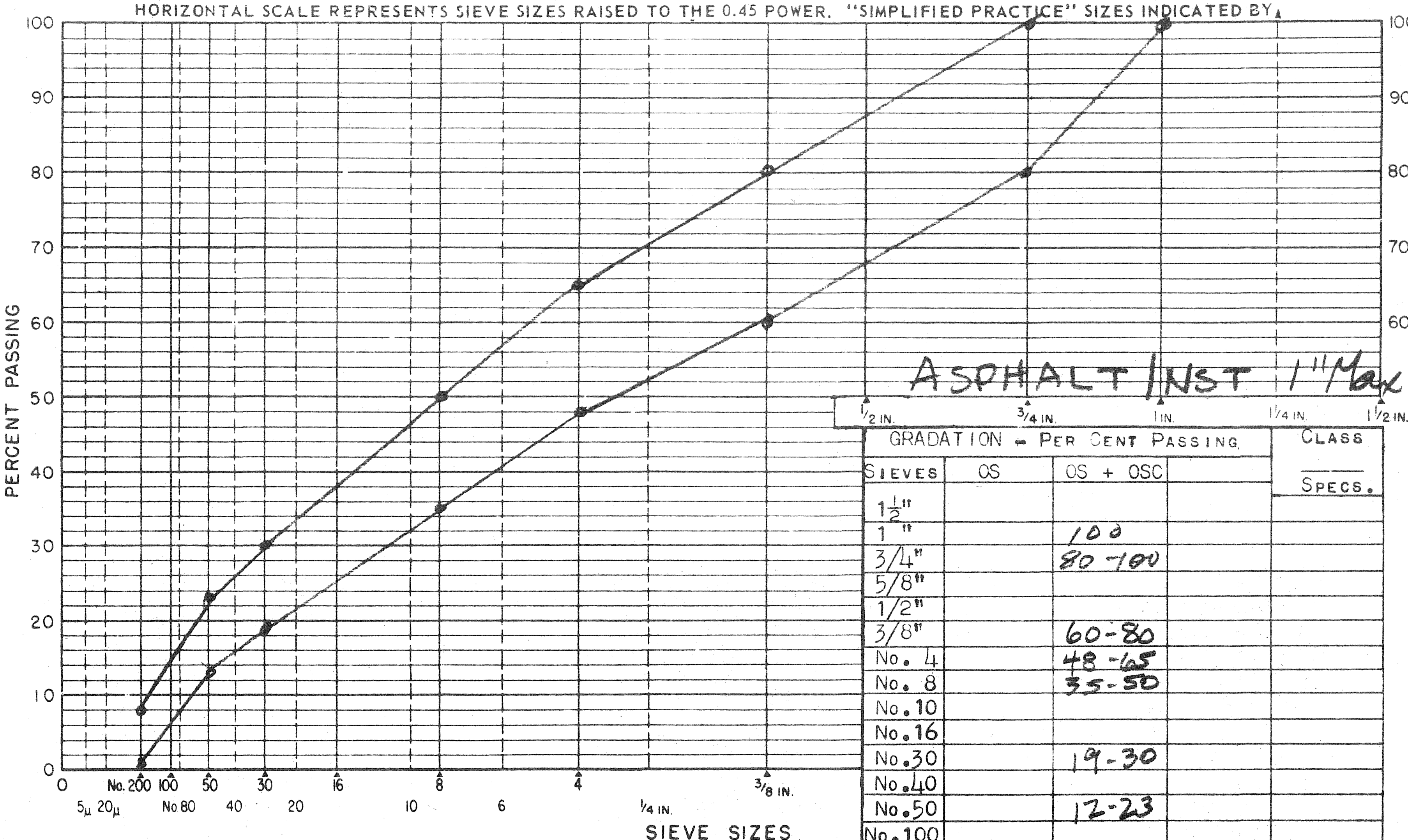
GRADATION - PER CENT PASSING				CLASS / SPECS.
SIEVES	OS	OS + OSC		
1 1/2"				
1"				
3/4"		100		
5/8"				
1/2"		80-100		
3/8"		70-90		
No. 4		50-70		
No. 8		35-50		
No. 10				
No. 16				
No. 30		18-29		
No. 40				
No. 50		13-23		
No. 100				
No. 200		4-10		

PROJECT ____________ LAB. NO. ____________

IDENT. No. ____________ PIT. No. ____________

GRADATION CHART

HORIZONTAL SCALE REPRESENTS SIEVE SIZES RAISED TO THE 0.45 POWER. "SIMPLIFIED PRACTICE" SIZES INDICATED BY ▲



GRADATION - PER CENT PASSING				CLASS
SIEVES	OS	OS + OSC		SPECS.
1 1/2"				
1"		100		
3/4"		80-100		
5/8"				
1/2"				
3/8"		60-80		
No. 4		48-65		
No. 8		35-50		
No. 10				
No. 16				
No. 30		19-30		
No. 40				
No. 50		12-23		
No. 100				
No. 200		1-8		

PROJECT ______ LAB. NO. ______

IDENT. NO. ______ PIT. NO. ______

SELECTION OF MATERIALS BASED ON TRAFFIC, CLIMATE, AND SUBGRADE SOIL

1. Bituminous Surfacing - The larger traffic volumes, ie., in excess of 3000 ADT, 300 Commercial, requires higher type pavements than do moderate or low volumes of traffic. As a guide, unless climate requires a higher type , I would suggest:

Traffic less 200 ADT (20-50 Comm) - 0.15' roadmix and a seal coat or a 0.15' plantmix with SC 3000 and a seal coat.

Traffic 200-1000 ADT (50-150 Comm) - a 0.20' Roadmix and a seal coat or a 0.2' plantmix and a seal coat. Use SC 3000 or 200/300 in plantmix.

Traffic 1000-3000 ADT (100-300 Comm) - a 0.25' plantmix including a seal coat.

Traffic 3000 + ADT (250 + Comm) - a minimum of 0.25' plantmix preferably a 0.30' minimum increasing to 0.4' at 5000 ADT.

2. Base Courses - Crushed granular stone to provide thickness required by subgrade or granular borrow. Granular borrow should be placed a minimum of 0.5' thick and should be considered as a portion of the structural section at 75 percent its thickness.

3. Climate - The zone maps are indicative of areas considered within each zone. There are areas overlapping all zones which should be evaluated individually. Soil types may enter into the decision as a clean granular subgrade may indicate dropping a zone whereas a frost susceptible silt in a high water table may require measures beyond even zone three requirements, ie., excavation and replacement to a depth of 2 or 3 feet below subgrade.

4. Subgrade Soils - Designs should be made on about the 80th percentile of the worst soil ie. lowest R-value or CBR's averages would indicate 50% under design and even the 80th percentile would indicate 20% under-

design. Use cross haul of better materials ie. granular, when possible and do not overlook grade point treatments. More frost heaves and failures occur at gradepoints than anywhere else. Drainage of any backfill at gradepoints is critical. Do not build into the grade any bath tubs to pond water.

In cross hauling soils, do not use silts or loams or any fine grained soils for improved subgrade unless it is used a minimum of 3 feet in thickness and design the structure for the soil used. Assure that if used it can be recognized visually be inspectors. Use of thin courses can act as a water trap to perch water in ase courses. Numerous failures have occurred where a few inches of silt was placed over quarry rock embankments due to perching water.

5. Rigid Pavement - Rigid pavement should be considered as an alternate to bituminous pavement. Economics generally will dictate choice although continuity of type is important from standpoint of maintenance.

Use of a treated base beneath a rigid pavement is used in lieu of dowels by California and many western states.

Reinforced pavements are gaining in popularity and continuously reinforced pavement is also gaining in popularity. The cost of reinforcing can be prohibitive when comparing to other choices. Idaho has one CRCP project and it is considered experimental. Other concrete pavements are of the plain (without dowels) over a treated base. The use of the variable spaced diagonal joint system is used. Plastic inserts to form joints has been used although sawn joints are also used.

Soils should be selected to provide a uniform subgrade throughout cuts and fills for rigid pavements. Transitions should be given careful consideration so as not to provide a frost heave or other problem.

On one Idaho job, all cuts were excavated 3 feet and recompacted to 100% T-99 density at ± 1% of optimum moisture to assure uniformity.

The soil in question was considered to be expansive having PI's in excess of 30.